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MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

AUSTRALIAN
INTERNATIONAL DEVELOPMENT
ASSISTANCE BUREAU

BRIDGE MANAGEMENT SYSTEM

BRIDGE INVESTIGATION MANUAL

FEBRUARY 1993

SMEC

SMEC - Kinhill Joint Venture

KINHILL

SNOWY MOUNTAINS ENGINEERING CORPORATION LIMITED

KINHILL ENGINEERS PTY LTD

BRIDGE INVESTIGATION MANUAL

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1. INTRODUCTION

This section introduces the Manual by defining its scope and objective, who should use the Manual, the structure of the Manual, and how it is to be used.

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This section of the manual gives details of the reconnaissance and topographic surveys required in bridge site investigation. Guidance is given on the type of data to be collected as well as the required extent and precision of the data.

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This section of the manual outlines the methods of soil exploration, lists the soil parameters required for design along with the appropriate testing method for obtaining each parameter and details a report format for presenting the results of the soil investigation.

10. FIELD TESTING

This section of the manual details the field testing methods required to obtain a quantitative assessment of the soils encountered. The design engineer is not expected to be able to carry out these investigations himself, but have a basic understanding of the methods in order to plan subsoil exploration and field testing programme.

12. DERIVATION OF DESIGN PARAMETERS

This section of the manual details the methods for computing the design parameters which are derived from the parameters obtained from field and laboratory testing.

APPENDIX A

Site Reconnaissance Questionnaire

This appendix includes a typical questionnaire which should be filled in by the design engineer carrying out a site reconnaissance so that all required data can be collected by a minimal number of visits to the proposed bridge site.

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BRIDGE INVESTIGATION MANUAL

SECTION 1

INTRODUCTION

BRIDGE MANAGEMENT SYSTEM
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1. INTRODUCTION

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1. INTRODUCTION

1.1 OBJECTIVE OF MANUAL

The objective of the Bridge Investigation Manual is to provide procedures for carrying out an investigation for a new bridge at a new or existing site, and procedures for checking the behaviour of an existing bridge, such as assessing its adequacy for the design flood and designing remedial scour protection works.

1.2 SCOPE OF MANUAL

The Bridge Investigation Manual describes procedures for bridge investigation which are performed prior to planning, designing and constructing a new bridge.

The Manual covers the fields of :

- preliminary investigation
- site surveys
- bridge type and site selection, and
- soil and waterways investigation.

The Manual includes a typical *site reconnaissance questionnaire* to assist the bridge design engineer to collect all the data necessary to complete the preliminary investigation.

The methods and procedures detailed in the Manual do not specifically require the use of a computer. However, the fields of *hydrology and hydraulics* include procedures which are more readily carried out by computer. The Waterways Investigation Section of the Manual therefore includes computerised methods.

1.3 STRUCTURE OF THE MANUAL

The Bridge Investigation Manual is divided into three Parts :

- Part 1 - General
- Part 2 - Waterway Investigation
- Part 3 - Soil Investigation

Part 1, General, covers the areas of preliminary investigation, site surveys, bridge type and site selection.

Part 2, Waterway Investigation, covers the areas of hydrology, hydraulics and scour prediction and scour protection.

Part 3, Soil Investigation, covers soil exploration, field and laboratory testing, and derivation of soil design parameters from the results of the testing.

Part 1 describes general investigation procedures, including use of procedures described in Parts 2 and 3, which are carried out during the course of a bridge investigation.

Parts 2 and 3 of the Manual are each self-contained and can be used without reference to any other part of the Manual.

1.4 WHO SHOULD USE THIS MANUAL

The Bridge Investigation Manual is intended to be used by qualified engineers for planning, investigation and design of new bridges and as a reference guide for checking the waterway capacity of an existing bridge and designing remedial works for scour protection including river training.

1.5 HOW TO USE THE MANUAL

Section 2 of the Manual describes the procedures for investigating a new bridge at a new or existing site. Following the procedures set out in Section 2, each Section of the Manual can be referenced in the sequence outlined for the particular type of investigation being carried out.

Specific Sections and sub-sections of the Manual can be used independently as follows :

- Part 2 of the Manual is used when the waterway capacity is required to be checked or remedial scour protection works must be designed.
- Part 3 of the Manual is used when a soil investigation program is to be carried out.
- Section 3 of the Manual is used when site surveys are to be carried out.

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BRIDGE INVESTIGATION MANUAL

SECTION 2

GENERAL

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2. GENERAL

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2. GENERAL

2.1 INTRODUCTION

This section of the manual outlines the purpose of, and procedure for, a bridge investigation for a new bridge at a new or existing site. It also details the preliminary office study and the investigation reporting required to be produced.

2.2 PURPOSE OF INVESTIGATION

2.2.1 General

The need for a road bridge arises when a road or highway is confronted with a natural or man-made obstacle such as a stream, river, ravine, canal, railway or another road where grade separation is required.

A bridge consists of a *superstructure* and a *substructure* consisting of abutments and piers. The superstructure or traffic-way of a bridge rests at each end upon substructures called abutments. Intermediate substructures are called piers.

The superstructure is generally made of steel, composite steel and concrete, reinforced or prestressed concrete or timber. The substructure is generally made of reinforced concrete.

The substructure is supported on *foundations* which may be either spread footings, piles or caissons.

Site investigation is an essential preliminary activity prior to the construction of all bridge works the purpose of which is outlined below.

2.2.2 Purpose

The purpose of a bridge investigation is :

- *Suitability*

  To assess the general suitability of the site and environs for the proposed works.

- *Design*

  To enable an adequate and economic design to be prepared, including the design of temporary works.

- *Construction*

  To plan the best method of construction and to foresee and provide against difficulties and delays that may arise during construction due to ground and other local conditions.

- *Effect of Changes*

  To determine the changes that may arise in the ground and environmental conditions, either naturally or as a result of the works, and the effect of such changes on the works, on adjacent works, and the environment in general.

- *Choice of Site*

  Where alternatives exist, to advise on the relative suitability of different sites, or different parts of the same site.

2.3 PROCEDURE FOR INVESTIGATION

The procedure for arriving at a final design for a bridge crossing over a river is a complex one in which structural, geotechnical and hydraulic factors are adjusted iteratively to achieve a bridge configuration which is satisfactory functionally, economically and aesthetically.

The extent of the investigation depends mainly on the size and nature of the proposed bridge works and the nature of the bridge site.

A bridge site investigation usually involves the following stages :

a. *a reconnaissance survey, including :*

   i. preliminary office study involving collection and study of existing information

   ii. an inspection of the site

b. *detailed examination for design, including :*

   i. site and catchment terrain

      - terrain evaluation
      - topographic survey

   ii. waterway investigation

      - hydrological studies

        - collection of hydrological data
        - delineation of catchment areas
        - estimation of design floods

      - hydraulic studies

        - estimation of peak discharges, water levels and flow velocities for the design flood

- waterway scour studies

  - prediction of bridge waterway scour
  - design of scour protection works

iii. soil investigation

- geological studies
- subsurface investigation and sampling
- field testing
- laboratory testing

2.4 RECONNAISSANCE SURVEY

2.4.1 Reconnaissance Survey Form

All information collected during the *reconnaissance survey* should be recorded on forms similar to that given in Appendix A of this *Manual*.

2.4.2 Preliminary Office Study

The first step in a bridge site investigation is a *preliminary office study* which entails collection and examination of all available records. Where there is a choice of site, information obtained from this study may well influence such a choice. Much information may already be available about a bridge site in existing records.

Information based on local experience, including earlier uses of the bridge site, may be available from local authorities and local or regional statutory undertakings. Occasionally, it is possible to obtain the results of previous ground investigations carried out on, or near, the bridge site. Excavations on, or near, the bridge site may have been made in the recent past by such authorities or with their knowledge. Information may also be available from local industry and others. Copies of old maps are often available in public libraries and local museums. Papers on local projects presented to professional bodies and local societies and technical journals may be other sources of information. Local oral tradition, although of variable reliability, may sometimes give a lead. Helpful information may be obtained from aerial photographs.

2.4.3 Site Inspection

The purpose of the *site inspection* is to gain a better understanding of stream behaviour by inspecting the channel boundaries, preferably at low stages of river flow. The information gathered will enable river behaviour due to changes in existing conditions to be predicted, hydraulic characteristics which will have an effect on the choice of general bridge design to be identified, and the extent of waterway training and bank protection works to be determined.

The *site inspection* should establish the following :

- type and grading of stream bed material

- existence of shoals and their composition
- the material forming the stream bank
- vegetation on the stream bank
- steepness of the stream banks and evidence of bank erosion
- erosion pockets and embayments in the stream bank
- existence of inerodible rock
- debris marks on shrubs, trees or banks which may indicate the water level of recent floods
- watermarks on walls, jetties and piers which indicate recent high-water levels.

When the assessment of the *reconnaissance survey* information has been completed acceptable sites for a bridge crossing from the *river morphology aspect* may be chosen.

2.5 SITE AND CATCHMENT TERRAIN

2.5.1 Terrain Evaluation

Terrain evaluation may initially be carried out in the office using :

- contour maps of the general catchment area
- aerial photographs in stereoscopic pairs

A visit to the area can then check the validity of the terrain evaluation from the contour maps and/or aerial photographs.

2.5.2 Topographic Survey

At each of the possible bridge locations to be investigated a *topographic survey* is required before further detailed investigation is carried out. The requirements for a *topographic survey* are given in Section 3 of this *Manual*.

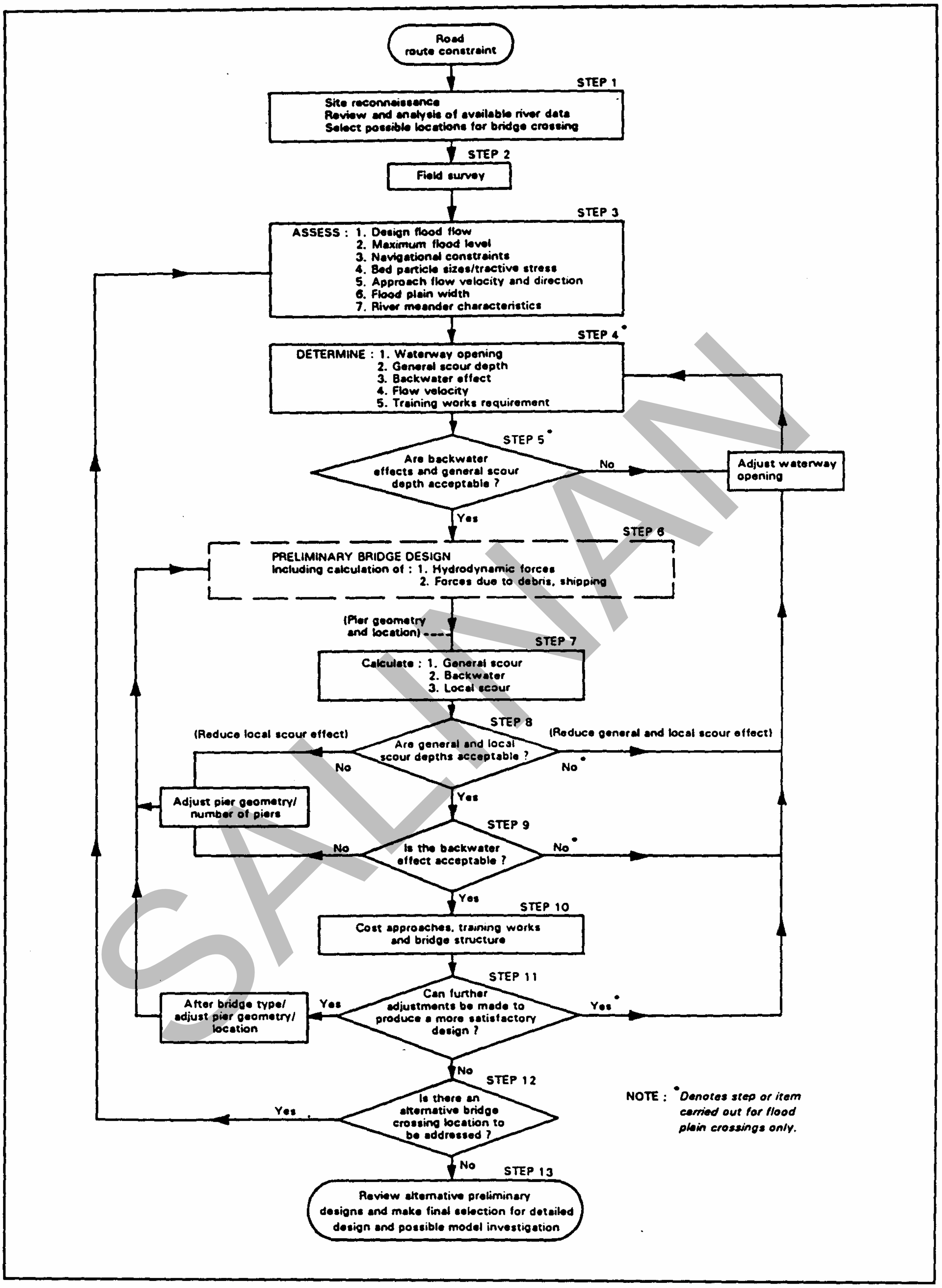

Figure 2.1 - Flow Diagram for Hydraulic Design of Bridge Waterways

2.6 WATERWAY INVESTIGATION

The step-by-step procedure for carrying out a bridge waterway investigation involves the iterative adjustment of the various hydraulic factors and is illustrated typically in the flow diagram of Figure 2.1 and further discussed in Table 2.1 below.

Table 2.1 - Procedure for Waterway Investigation

Step	Procedure for Waterway Investigation - (Table 2.1)
Step 1	The *reconnaissance survey* and the *review and analysis of available river data* will enable a selection to be made of possible bridge locations which are compatible with the proposed route of the road.
Step 2	At each of the possible bridge locations the topographic survey will be carried out.
Step 3	From the available data the following parameters will be assessed : • design flood flow • maximum flood level • navigational constraints on bridge height and pier locations • approach flow velocity and direction • flood plain width • river meander characteristics Where it is necessary to make an appraisal of the consequences and cost of the design discharge being exceeded, measured against the additional capital cost of a bridge designed for a flood of a longer return period, the design procedure may be repeated from this step for flood discharges of various return periods.

Step	Procedure for Waterway Investigation - (Table 2.1)
Step 4	The following will be determined within this step of the design process. *a. Waterway width* For bridge crossings over flood plains, the option to confine the river width should be considered. A confined river is one which at the design discharge, flows at a width equal to, or less than, the regime width. An unconfined river has substantial flood plain flow at the design discharge. In rivers with meandering channels it is usually cheaper to confine the waterway opening and make the crossing in a combination of embankment and bridge construction than to bridge the full width of the flood plain. A trial waterway opening may be obtained as outlined in Section 6, *Hydraulics*, of this *Manual*. The total bridge span may be obtained from the waterway width by making allowance for the obstruction to the flow of the piers and the skew of the bridge to the principal direction of flow. An intuitive allowance for obstruction to the flow by the piers can be made at this point which will be made if the waterway opening is measured normal to the principal direction of flow. The effect of decreasing the waterway opening will be to increase the depth and velocity of flow and to make the backwater effect more severe. The spanned length will be correspondingly shorter but the foundations may need to be deeper and must be capable of resisting larger hydrodynamic forces, and the rip rap protection of the guide banks will need to be stronger. Also, depending on the backwater effect, dyke or storage pond construction to prevent overtopping of the river banks upstream may become necessary. It is therefore evident that the economies achieved by decreasing the waterway opening and shortening the spanned length may be offset by increased costs for the foundation and training works, and possibly by increased damage due to raising of flood levels. *b. General scour depth* The average depth of general scour in a confined waterway may be calculated from Section 7, *Scour Prediction*, of this *Manual* according to whether the channel bed is of sand or of gravel, or of a cohesive material. For an unconfined waterway the dominant discharge or bankfull discharge may be used to determine the general scour depth. Alternatively, field measurements of the river channel geometry may be used to assess general scour depths.

Step	Procedure for Waterway Investigation - (Table 2.1)
Step 4 (contd.)	*c. Backwater* The backwater effect due to a reduced waterway opening may be calculated at this stage and adjusted for the effects of general scour by the method given in the Section 6, *Hydraulics*, of this *Manual*. A more refined calculation, which will use the pier geometry and location obtained from the preliminary bridge design, will be carried out later in Step 9. *d. Flow velocity* The flow velocity for incised and unconfined rivers will have been assessed from the field measurements described in the previous step. For a confined waterway, the average velocity may be calculated from the design flow, the width of the waterway and the average general scour depth. The maximum channel velocity may be obtained by factoring the average velocity by the multipliers given in Table 8.4, Section 8, *Scour Protection*, of this *Manual*. *e. Training works* The requirement for training works will depend on the stability of the approach channel, on whether the waterway opening is being confined and on the nature of the river bank material. In situations where groynes and guide banks are necessary, they will invariably require protection in the form of rip rap. The availability of suitable rock is therefore an important factor when considering the cost of training works. In regions where rock is not readily available, crossing locations which require a minimum of training works will have obvious advantages and, in extreme situations, crossings which bridge the full width of the flood plain may become cheaper to construct than those which confine the waterway openings and require guide banks. Section 8.4 (Section 8, *Scour Protection*) of this *Manual* gives guidelines on river training works.
Step 5	At this stage the general scour depths and backwater effect may be reviewed. If general scour depths are such that foundation depths for the bridge and training works are too great, or unacceptable impounding of flood water occurs, or the river cannot be contained within the existing banks by a reasonable amount of training works, then it will be necessary to adjust the waterway opening and return to Step 4.

Step	Procedure for Waterway Investigation - (Table 2.1)
Step 6	*a. Influencing factors* In this step in the design procedure, the various alternative types of construction for each possible crossing location will be considered. Their effect on the river regime will be considered and the consequential design requirements, as illustrated in Table 2.2, will be assessed. Table 2.2 is for crossings over meandering dynamically stable channels, but may also be used as a guide in design of crossings over straight and braided river channels. Construction types which are unsuitable for the prevailing conditions will be eliminated, and those which are suitable will be developed to a stage where their cost may be evaluated for comparison purposes. Many of the factors which will be taken into consideration in the design of a bridge crossing over a river will be common to other types of bridge crossing. These will include : • structural loading (deadweight, wind, etc.) • ground conditions • economy of construction • the availability of plant, material and skilled labour • the prevailing climate • access to the site • environmental impact • future maintenance • specific requirements of the Province Certain factors apply particularly to crossings over rivers, and can affect the bridge configuration. These are discussed below. *b. Height of bridge* In cases other than of the submersible bridge, which is specifically designed to be overtopped for a limited number of days during the year, the clearance between the underside of the bridge and the maximum flood level will be dictated either by navigational requirements or, in a river reach where there is no river traffic, by the freeboard necessary to ensure free passage of debris. The freeboard allowance adopted will largely depend on the tree and vegetation growth on the river banks upstream. In Indonesia a 1.0 metre freeboard is required to the underside of the deck beams. In cases where the freeboard has been kept to a minimum, it may be prudent to check on flood levels for floods with longer return periods. Should the bridge be submerged under these conditions then the effects of uplift and hydrodynamic forces on the bridge deck should be checked.

<table>
<tr><th>Step</th><th>Procedure for Waterway Investigation - (Table 2.1)</th></tr>
<tr><td>Step 6
(contd.)</td><td>It should be noted that in some cases the geometric constraints imposed by the bridge approaches will require a minimum bridge level which may be above the minimum level determined from freeboard considerations.

c. Pier geometry

Pier geometry should be selected after taking due consideration of :

• superstructure loading (deadweight, wind, etc.)
• wind load
• type of foundation
• hydrodynamic forces
• impact forces (debris, ship)
• direction of river flow to pier alignment
• local scour
• backwater effect
• aesthetics

The geometry should be such as to minimise backwater effects and scour. Piers should therefore be aligned with the principal direction of flow and present as small a projected area to the flow as structural and aesthetic considerations allow.

d. Pier location

Pier location will be dependent on :

• the requirement for safe navigation and the degree of protection required against ship impact
• the economic span length for the type of bridge construction under consideration
• ground conditions
• method of foundation construction
• channel geometry
• backwater effect
• aesthetics

Under certain circumstances the channel geometry can have a significant influence on the type of bridge construction considered and therefore on the pier locations. Where, for example, a type of bridge construction may be selected which spans the deep channel completely, so avoiding costly and difficult construction within the channel and possible problems due to the effects of local scour.</td></tr>
<tr><td>Step 7</td><td>The proposed bridge configuration will be used in this step in the calculation of the general and local scour depths and the backwater effect.</td></tr>
</table>

Step	Procedure for Waterway Investigation - (Table 2.1)
Step 8	The influence of the combined effects of general and local scour in the waterway on the design of the pier foundations will be checked in this step. If a relatively small reduction in scour is required to improve the foundation requirements, then local scour (and in incised river crossings, general scour to a small extent also) may be reduced by adjusting the pier geometry or the number of piers, i.e. returning to Step 6. If a greater reduction in scour is required, then the local and general scour effects will need to be reduced by adjusting the waterway opening, i.e. returning to Step 4.
Step 9	The backwater effect due to obstruction of the flow by the piers will be calculated in this step. For the incised river crossing this will be the first backwater calculation. For the flood plain crossing it will be a refinement on the calculation carried out in Step 4. If the backwater effect proves excessive, then in the case of the incised river, adjustment to the pier geometry and to the number of piers obstructing the flow will be necessary, i.e. returning to Step 6 and in the case of the flood plain crossing, adjustment to the waterway opening will be necessary, i.e. returning to Step 4.
Step 10	At this stage the design will have been advanced sufficiently for the cost of each of the preliminary bridge designs to be assessed. For confined flood plain crossings the total cost will include the cost of the approach embankments within the flood plain, as well as that of the training works and the bridge structure.
Step 11	The costs of the alternative schemes for each crossing location will be appraised in this step. If the cost of a scheme is outside the budget allocation, then savings may be possible either by altering the bridge construction by returning to Step 5, or by making adjustments to the waterway opening by returning to Step 4.
Step 12	If there is an alternative location for the bridge crossing, then the design process is repeated from Step 3.
Step 13	The alternative bridge designs for each of the alternative bridge crossing locations will be reviewed and the best scheme selected for detailed design.

Table 2.2 - Effect of Bridges on Meandering Dynamically Stable Channels

Structure	Effect	Result	Design requirements
Embankment	Obstruct drainage in flood plain and increase flow intensity through opening	Local scour at piers and abutments	Increase size of pier and abutment and their foundations Apron/mattress to limit depth of scour
		Increase hydrodynamic force on piers	Increase size of pier and foundations
		Increase upstream water level, and magnitude and frequency of floods upstream	Dyke construction or flood storage
		Local bank erosion	Bank protection
	Obstruct migrating meander	Extensive bank erosion downstream	Bank protection River training upstream
		Scour at toe of embankment on upstream side	Toe protection
Abutment	Obstruct migrating meander and change pattern	Extensive bank erosion downstream	River bank protection downstream River training upstream
	Defect flow pattern and increase local flow intensity	Local scour at abutments	Increase abutment depth or apron/mattress to limit depth of scour
		Local bank erosion downstream	Bank protection
	Reduce width of waterway and increase flow intensity through opening	Increase scour at abutments, piers and in waterway	Increase depth of abutment and piers or apron / mattress to limit depth of scour
		Bank erosion downstream	Bank protection downstream
		Increase upstream water level and magnitude and frequency of floods upstream	Dyke construction or flood storage
Pier	Deflect flow pattern and increase local flow intensity	Local scour at piers	Increase size of pier and foundations or apron / mattress to limit depth of scour
		Increased hydrodynamic forces on piers	Increase size of pier and foundations or apron / mattress to limit depth of scour
	Reduce width of waterway and increase flow intensity through opening	Increase scour at pier, abutment and in waterway	Increase size of pier and foundations or apron / mattress to limit depth of scour
		Increase upstream water level and frequency of floods upstream	Dyke construction or flood storage

2.7 SOIL INVESTIGATION

Once a bridge site is finally selected, a comprehensive *soil exploration and testing* program should be undertaken as outlined in Sections 9 - 12 of this *Manual*. However, if there is some doubt about the subsurface conditions during the assessment of alternative bridge sites then some preliminary *soil investigation* may have to be carried out at each site in doubt.

2.8 INVESTIGATION REPORTING

2.8.1 General

The fundamental purpose of a bridge site investigation is to obtain information about the site such as topography, geology, soil design parameters, hydrology and waterway characteristics to enable the design engineer to plan, design and construct a new bridge. At the completion of a *bridge site investigation* a complete and comprehensive report should be prepared that places the work done in context and effectively communicates the results of the investigation.

While all bridge site investigation reports should conform to normal good writing practice, there are several features peculiar to reports on site investigations and these should receive particular attention. These features are briefly outlined below.

2.8.2 Reasons for Preparing Report

The reasons for preparing a bridge site investigation report are :

- To communicate the results of the bridge site investigation.
- The systematic assembly of material for the report will ensure that no important aspect of the site has been overlooked.
- A detailed report is usually the necessary pre-requisite for the detailed design and construction of the bridge.
- The report will be a record for use in the future for :
    - basic data to enable intelligent decisions to be made in case of any defect or unforseen condition developing during the life of the bridge
    - reassessing the competence of the bridge works in the light of future developments in technology
    - planning any future developments for the bridge and bridge site.

2.8.3 Completeness of the Report

All aspects of the *bridge site investigation* should be systematically reported. Care should be taken to avoid the tendency to report only the unusual features found during the investigation. In particular, when a possible hazard or deficiency has been investigated and found to be non-existent this should be reported or the value of the work done will be lost because others, not being certain, will check this same point. Liability to flooding is an example.

Sufficient outline of the reasons for the bridge site investigation, the bridge works envisaged at the time, who requested the site investigation, and who carried it out should be included in the report. In addition to providing necessary background to the current designers, this information is often invaluable in the future in assessing the validity of using information contained in the report for purposes of which the original investigators were quite unaware and hence gave no consideration.

2.8.4 Identification of the Bridge Site

In most cases it can be expected that not too long after the *bridge site investigation* is completed the site will be significantly altered by construction operations. It is therefore desirable that the site investigated should be adequately defined by reference to surveyed monuments, property lines, or an established coordinate grid, and not features which could be removed.

2.8.5 Factual Data and Interpretation

Throughout the report the clear distinction should be made between what is factual data, such as observations, test results, etc., and what are interpretations and assessments made from the data.

2.8.6 Information on Construction Drawings

On the basis of information gathered during *bridge site investigation*, the designer of the bridge works to be constructed on the site will assume that certain conditions exist in the ground. If the actual conditions found during excavation are different to those assumed, it is essential that the bridge designer be informed of the difference immediately.

To enable bridge site personnel to be aware if differences do occur, it is recommended that a sufficient description of the strata to be expected during excavation be included in the drawings and documents handled on the bridge site during construction.

2.9 REFERENCES

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DIRECTORATE GENERAL OF HIGHWAYS
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REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 3

SITE SURVEYS

BRIDGE MANAGEMENT SYSTEM
BMS
A PROJECT JOINTLY FUNDED BY INDONESIA AND AUSTRALIA

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3. SITE SURVEYS

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3. SITE SURVEYS

3.1 INTRODUCTION

This section of the manual gives details of the reconnaissance and topographic surveys required in bridge site investigation. Guidance is given on the type of data to be collected as well as the required extent and precision of the data.

3.2 GENERAL

The design of a bridge over a stream requires that detailed attention should be paid to the route location, potential traffic flow, structural and foundation requirements as well as to the characteristics of the stream flowing beneath.

Therefore it is necessary to collect information on the bridge site topography, geology, soil design parameters, hydrology, and waterway characteristics.

Information is collected by means of Reconnaissance Survey and Topographic Survey. Details of the requirements of these surveys are given in the following sections of this manual.

3.3 RECONNAISSANCE SURVEY

3.3.1 General

A Reconnaissance Survey is the first field inspection of a proposed bridge site to collect data for assessing the suitability of the site. It also allows further decisions to be made about carrying out a topographic survey, a soil investigation and a hydraulic / hydrological study in order to be able to perform a good bridge design.

3.3.2 Reconnaissance Survey Procedure

The following factors must be considered in carrying out a bridge Reconnaissance Survey.

a. Bridge Location

The selection of a bridge site should take into account :

- cost
- social aspects
- aesthetics of the bridge and road alignment
- simplicity of design and ease of construction

If there is a need to relocate the existing bridge, then consideration must be given to land acquisition, the possibility of deep excavations which may affect the adjacent building structures and other factors which may cause problems during construction.

b. Span, Width and Type of Bridge

Selection of the bridge span length, width and bridge type should take into account the stability of the abutments, the river profile, direction of river flow, river characteristics, river sedimentation material, scour behaviour, the volume of traffic and design loading on the bridge.

The design of bridge approaches on swampy or soft soil or fill material may create stability or settlement problems. This may necessitate increasing the bridge span length or stabilising the approach embankment foundations to avoid excessive settlement.

c. Hydraulics/Hydrology

Collection and evaluation of hydraulic and hydrological data and river morphology characteristics is necessary to assess the river flow characteristics at the proposed bridge site.

The date of occurrence and level of the highest recorded flood should be collected and return period estimated for comparison with the design flood return period.

In order to estimate design floods, streamflow or rainfall data may be obtained from *Departemen Pekerjaan Umum, Balai Penyelidikan Hidrolika*.

d. Soil Investigation

The approximate location of the bridge must be selected so that a soil investigation can be carried out (boring, Dutch cone penetrometer, Standard Penetration Test, test pits etc.).

For the determination of the type of substructure for the new bridge, the soil investigation data from the previous bridge may provide the necessary information.

e. Previous Bridge Data

The load carrying capacity of the existing bridge must be assessed if it is to be used as a temporary bridge while the new bridge is being built.

An assessment should be made of the effect of the previous bridge on the river characteristics to ensure that it has no detrimental effect on the new bridge. The previous bridge may have to be demolished if it obstructs the waterway significantly or creates excessive scour adjacent to the new bridge.

f. Quarry Sites

Information must be collected on local quarry areas for quality of materials and distance from the proposed bridge site. The local DPUP staff should be able to help with such information.

g. Reconnaissance Survey Staff

Experienced reconnaissance survey field staff must be used in order to obtain an good survey with accurate assessment of site conditions.

h. Photos

Photos are required of the existing bridge and proposed new location. These photos can be used later for assessing the existing bridge condition, characteristics of the river channel, characteristics of the surrounding area and characteristics of the proposed new location.

These photos should include :

- photos of the proposed new location

- looking downstream
- looking upstream
- looking at each abutment
- perspective view of the site
- other photos which may assist during design

These photos should be annotated with river flow direction, the axis of the proposed new bridge, abutment location etc.

- photos of the existing bridge

3.3.3 Reconnaissance Survey Report

The results of the reconnaissance survey should be presented on a report form similar to that in Appendix A of this manual. The completed report form must be signed and also include the original photos taken at the bridge site.

3.4 TOPOGRAPHIC SURVEY

3.4.1 General

Bridge Topographic Survey is one of the activities in the bridge design process which produces detailed topographic survey drawings of the bridge site and surrounding area. It is carried out after the Reconnaissance Survey has been completed.

The purpose of the Topographic Survey is to collect survey data to enable accurate layout and design of the new bridge.

3.4.2 Topographic Survey Procedure

The topographic survey is carried out along the alignment of the proposed new road and bridge to a width which will allow re-alignment of road and bridge if required without carrying out another survey.

The survey should include the following :

a. Survey at Bridge Site

- establishment of horizontal and vertical control points
- survey of the existing bridge site
- survey for longitudinal and transverse sections
- installation of survey posts
- preparation of survey plans including layout coordinates
- survey along the re-aligned bridge axis

b. Survey of Waterway and Area in Vicinity of Bridge Site

- on the left and right side of the river along the road axis for a distance of 200 metres. The survey width at the left and right of the road each side is 50 metres.

- on the left and right at the river channel for a distance of 100 metres. The survey width at the left and right from the river banks each side is 50 metres.

c. Survey Method

i. Survey Control Points

- **Horizontal Control Points**

  - establishment of control points can be either by a net of traverse points or triangular meshes. The choice of the control point type depends on the river width. For rivers wider than 100 metres the triangular mesh method shall be used.

  - control points shall be placed 50 - 100 metres apart.

  - for traverse net and triangular mesh, Level II survey instruments shall be used.

  - a minimum of 6 concrete posts shall be installed for the river and bridge site survey (2 at a distance of 500 metres on the left and right of the river and 4 at the bridge site).

  - sun azimuth measurements should be carried out at the bridge site.

- **Vertical Control Points**

  - levelling instruments shall be used to carry out 2nd order measurements.

  - elevation levelling crossing a river shall use the *double line crossing* method for rivers wider than 75 metres.

  - control points at 50 m distances shall be made of concrete

  - the elevation of the control points shall be fixed to a known elevation point.

  - survey accuracy :

    * the height difference measurement shall be measured forward and backward. The height of the polygon shall be measured using a 2nd order measuring instrument.

    * this height measurement shall be fixed to an existing height point of which its mean altitude above sea level is known.

- the average error of this levelling shall not exceed 1.5 - 2.5 km : total road length being measured.

d. Survey at Bridge Location

Survey along the bridge axis and adjacent area shall cover all existing features along the roadway such as houses, trees, the edge of the road and shoulder, location and dimensions of ditches and culverts, electric poles, telephone poles, bridges, edges of paddy fields, gardens, borders of villages, rivers, irrigation channels, direction of water flow etc.

For this survey the tachymetry method shall be used.

- the coordinates of the Km and Hm posts at the edge of the road shall be taken and calculated. This is to increase the reference points for locating the proposed road axis.

- at quarry locations, the access road shall be marked on the map as well as the type of material and its location.

e. Survey at Sections

i. Survey of River Cross-Sections

In the river channel, cross-sections shall be made every 25 metres up to a distance of 100 metres on the left and right of the roadway axis. The width of the cross-sections shall be 50 metres to the left and right from the river bank or bridge abutment.

ii. Survey of Road Sections

- **Longitudinal Sections**

Survey of the longitudinal section shall be along the existing roadway axis. At places where re-alignment may occur, additional section shall be made.

For survey of the longitudinal section the same instruments shall be used as for surveying the vertical control points.

- **Cross-Sections**

Survey of transverse cross-sections shall be made every 50 metres at straight and flat roads and every 25 metres for curved and undulating roads.

The survey width shall cover an area 50 metres to the right and left of a straight road and 25 metres outward and 75 metres inward on a curved road.

Features to be survey include the edge of the pavement, invert and obvert of culverts, edge of the road shoulder, top and invert of ditches, irrigation channels, elevation of bridge deck and river banks.

The instruments for carrying out this survey are the same as those used for cross-sections.

f. Survey Posts

Concrete posts with a size of 10 x 10 x 75 cm shall be embedded is such a way that only 10 cm of the post shall be above the ground.

Traverse and section posts shall be made of wood with a size of 5 x 7 x 60 cm.

Concrete and wooden posts shall be marked with BM and numbered consecutively.

To increase the number of elevation fixed points, it may be necessary to place reference elevation points at trees or other permanent locations which are easily found again later.

Traverse and section posts shall be marked with yellow paint and red figures placed to the left of the survey direction.

For longitudinal sections, the point located on the road axis shall be marked with nails and circled with yellow paint.

g. Computation and Mapping

Coordinates of the primary traverse points shall be calculated and correlated to the fixed points being used.

Calculation shall be based on the least squares method.

Plotting of traverse points shall be based on coordinate calculation. Traverse point plotting shall not use a graphical method.

3.4.3 Topographic Survey Report

The bridge topographic survey report shall consist of :

- A survey map plotted on millimetre paper with a scale of 1:500 and elevation contours of 0.25 metres. Elevation of detail points shall be shown on the survey map along with any important notes.

  Fixed points and new fixed points shall be plotted on the survey map with special marks. The elevation of these points shall also be recorded.

- Coordinates and elevations of the primary traverse points shall be appended to the topographic survey report.

3.5 REFERENCES

Indonesian Language References

3.1 Departemen Pekerjaan Umum, Direktorat Jenderal Bina Marga, Direktorat Bina Program Jalan, *Survai Pendahuluan Jembatan*, 1980.

3.2 Departemen Pekerjaan Umum, Direktorat Jenderal Bina Marga, Direktorat Bina Program Jalan, *Survai Topographi Jembatan (Penggukuran Jembatan)*, 1980.

DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 4

SITE SELECTION

BRIDGE MANAGEMENT SYSTEM
BMS
A PROJECT JOINTLY FUNDED BY INDONESIA AND AUSTRALIA

FEBRUARY 1993

DOCUMENT No. **BMS5-M.E**

4. SITE SELECTION

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4. SITE SELECTION

4.1 INTRODUCTION

Scope

This section of the manual discusses the basic principles of selecting a suitable bridge site prior to proceeding onto a detailed site investigation.

Definition

The word *bridge* will be considered in a broad sense to include all kinds of crossings over water including both bridges and culverts. This manual will deal mainly with bridges over water, but the same principles apply to grade separation structures, to structures over or under railways, etc. One major difference between these is that there is no need to consider hydraulics for the latter types of structures, the size of the structure being determined by minimum clearance requirements.

Procedure

The process of selecting a suitable bridge site is a step by step procedure, with information being collected in the field and then analysed in the design office. This cycle of field work followed by office work may have to be repeated several times. It is therefore important for the bridge designer working in the office to have a check list of items (refer to *Bridge Design Manual*) for which data is required for the final detailed design of the bridge. However, if there are different persons doing the office work and the field work, good communication must exist between them. In many instances the office engineer must go out on site to have a personal appraisal of field conditions.

Considerations

In selecting a bridge site, a number of factors must be considered. The main factors to be taken into account are :

- bridge alignment
- type of crossing
- soil investigations
- costs

In many cases, it is impossible to satisfy all the requirements, the bridge designer can only select the best solution.

4.2 BRIDGE ALIGNMENT

The general principle to be followed is that the bridge should be square, that is, at right angles to the obstacle (for example, a river) and it should be as short as practicable. Figure 4.1 compares a square alignment with a skew one.

Figure 4.1 shows that the length and hence cost of the skew solution will obviously be greater than the square alternative. However, the assessment is not as simple as this. It is important to consider the bridge as part of the road. Thus the structure must satisfy the geometric road design standards for the facility it carries and hence the geometry of the structure will be

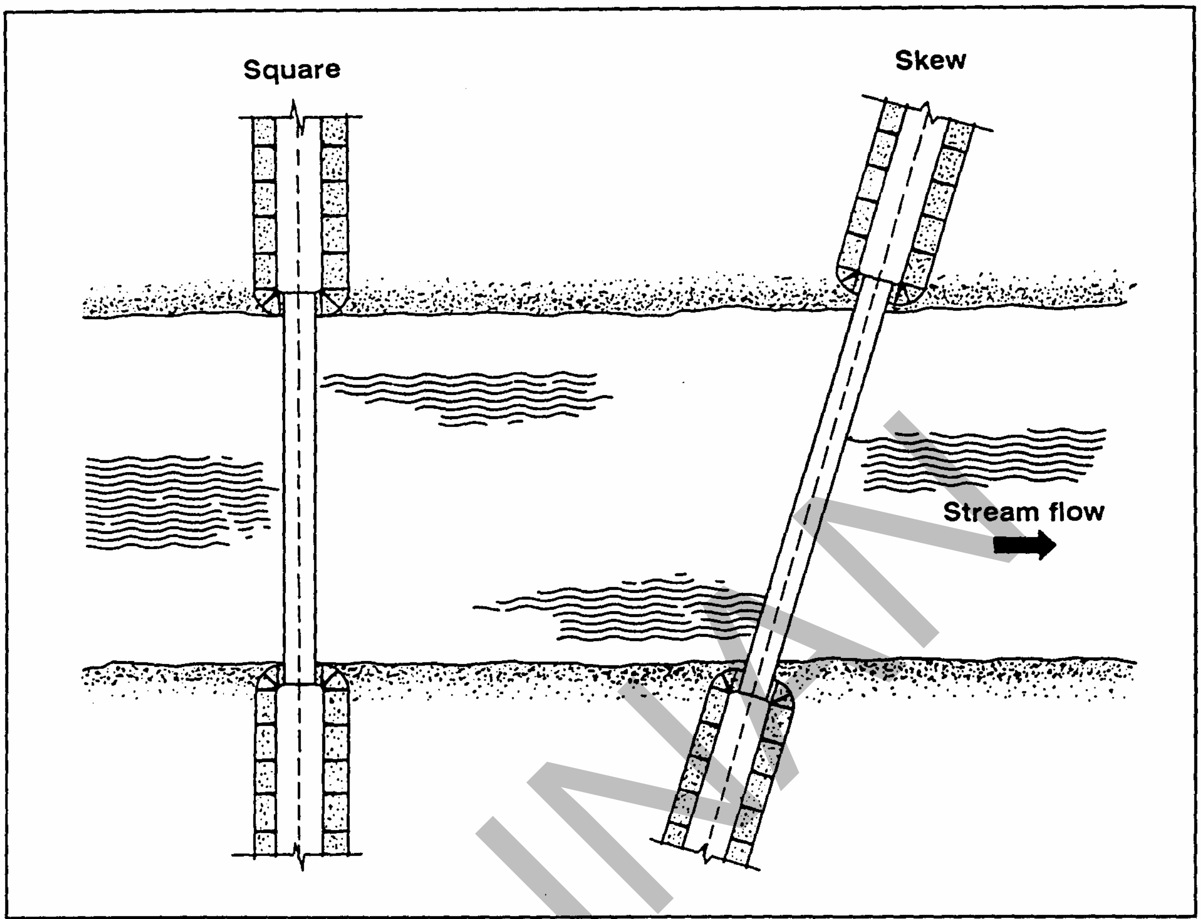

Figure 4.1 - Square versus Skew Alignment

governed by the function of the road. All this simply means that the simplistic comparison of Figure 4.1 is not always true. For instance, in Figure 4.2, Alternative B will be, in most cases, better than Alternative A.

An acceptable bridge site is then one for which the bridge and approaches are entirely satisfactory from the point of view of road design. It should be pointed out again that an increasing number of structures are being determined by road grading requirements which overrule other factors such as waterway requirements. It is therefore necessary for the bridge designer to have an understanding of the basic principles of road location and design. Of course, the road designer must also be aware of the requirements of the bridge designer.

In most cases, then, the alignment of a bridge will be settled through discussions between the road designer and the bridge designer. Both sides must compromise to reach a speedy and realistic solution. The relative importance of the road alignment and bridge are compared on the basis of overall cost and benefits. In many cases, the demands of traffic volume and safety dictate the location ana alignment of the structure. In some instances, the road alignment can be varied on the grounds of bridge economy, especially so in the case of major structures in rural areas.

The process in which a tentative road alignment, and hence bridge location, is selected

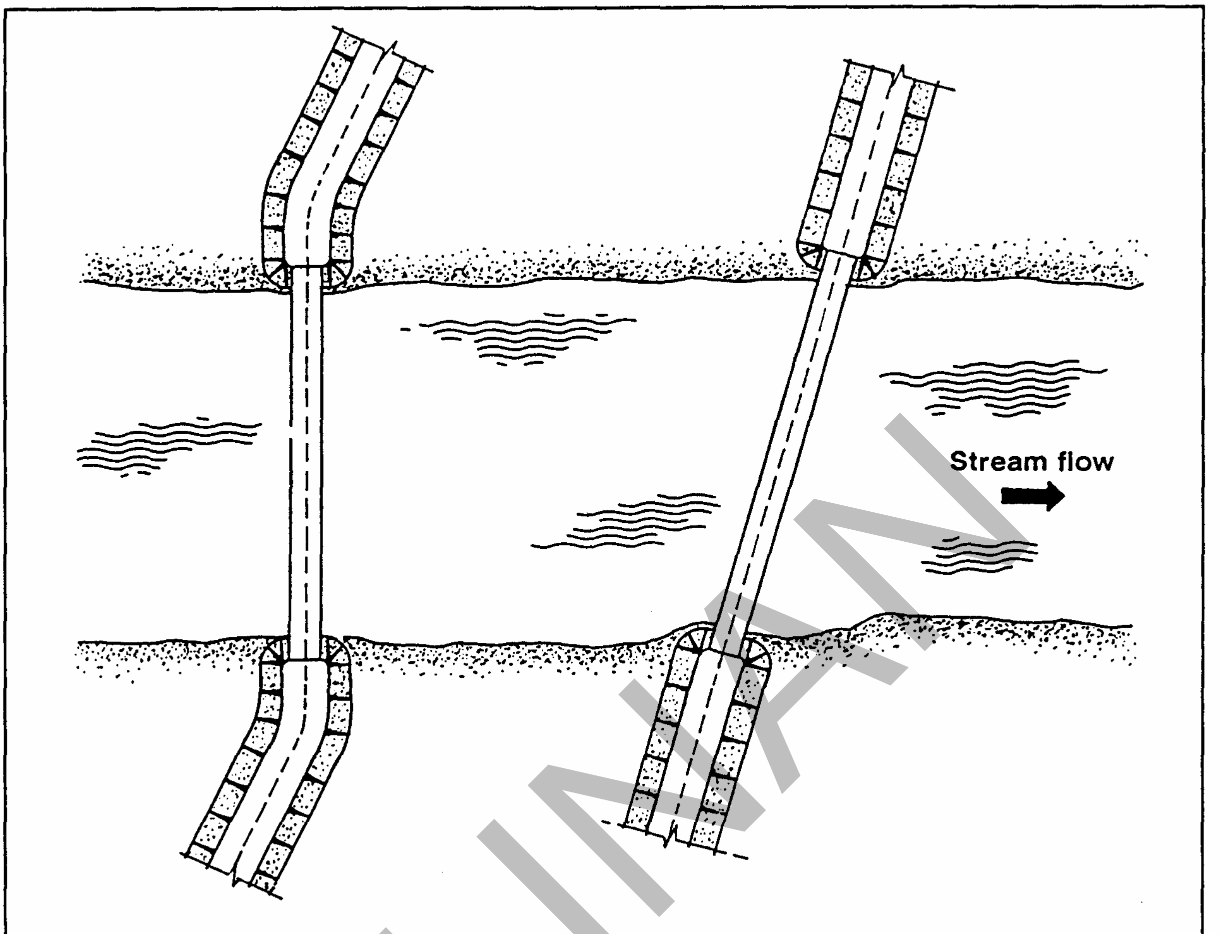

Figure 4.2 - Consideration of Bridge and Road

involves usually a submission of alternative alignments by the road designer to the bridge designer. Each of these alignments is feasible from the point of view of road design standards and economy. The bridge designer will then look at these alternatives from the point of view of bridging and make a recommendation. Other factors that need to be considered are detailed in the following sections. The designer, however, needs not look at these factors in detail. A complete and thorough assessment will be done once the general problem of location has been solved, and a bridge type has to be selected.

4.3 TYPE OF CROSSING

4.3.1 Crossing Suitability

The suitability of each alternative crossing put forward as a possible bridge site will depend on the type of the crossing including the river characteristics, where applicable.

Where a river crossing is involved, it is important to make a relatively detailed study of the waterway requirements at an early stage of the investigation. This study should cover the magnitude and frequency of floods, flood levels, stream velocities, position of river bed and general hydraulic behaviour at each possible crossing site.

An idea of the type, height and length of bridge required should then be obtained for each crossing considered.

4.3.2 Bridge Types

Bridges are usually classified into four broad types depending on the relationship between the flood levels and the deck levels of the structure. These are (see Figure 4.3) :

- **High Level Bridge**

  A high level bridge is where the deck of the structure and of the approaches are flood free for the design flood. This is usually the most expensive structure.

- **Low Level Bridge**

  A low level bridge is where the deck of the structure is above the normal flow of the stream but submerged by the design flood. This type of structure is usually adopted for reasons of economy. It is suitable proposition for dry areas, where large floods occur rarely or in mountainous country where floods could be frequent but would be of short duration.

- **Fords**

  Fords can be in the form of a paved crossing of the river bed which would be safe from scouring, possibly a concrete slab. In normal flow water passes over the slab at very shallow depth.

- **Floodway or Causeway**

  A floodway or causeway, on the other hand, is constructed on a slightly higher level than the stream bed. Very often a number of pipes or other types of openings are provided under the causeway to take dry weather flow. A floodway can be expected to be available for use by traffic for a greater proportion of the time than a ford but is usually more expensive.

The type of structure adopted for any particular crossing depends on the funds available and on the importance of the road on which the crossing is situated. In general, the saving in cost has to be weighed against the economic losses caused by the interruption to traffic.

4.3.3 Site Conditions

As expected, for each of the above types of bridge there are certain conditions which should be sought in the selection of the site. Some of these conditions for each of these types are :

High Level Bridge

- Narrow, deep crossing allowing a square bridge.

- Stream bed should be free from scour and siltation.
- Broad flood plains or branching streams are undesirable since distribution of flow is difficult to calculate and it varies from one flood to another.
- Suitable foundation, for example, rock at shallow depth or firm materials for economical piling.

Low Level Bridge

- Flood plain situations are acceptable.
- Narrow deep channels and not usually suitable unless the banks are cut. In many cases, siltation then becomes a problem.
- Ideally, a broad and shallow stream bed with gently sloping banks is required. It is even more important to prevent scour and siltation in this case than for a high level bridge.

Fords and Floodways

- Broad, shallow and reasonably level stream beds are required.
- The stream bed must be stable.

Floodways can be used in conjunction with pipes or other types of culverts which will pass the dry weather flow (Figure 4.3d).

Culverts

In many instances, it is more economical to use culverts only instead of bridging. The bridge designer must always keep this in mind since culverts can be the best solution. A few relevant points regarding the use of culverts are listed below.

- May be better than bridges in steep country requiring high fills for bridges provided waterway requirements are satisfied.
- Useful for part-width road construction.
- Useful where the bridge geometry becomes too complex, for example, short radius road curves, particularly in combination with skew crossings and vertical curves.
- Pipe culverts can be economical in isolated sites.
- Should not be used where debris is possible, where the foundation material is soft, or where extensive stream bed excavation is necessary.
- Permanent water at the site can also be a problem.

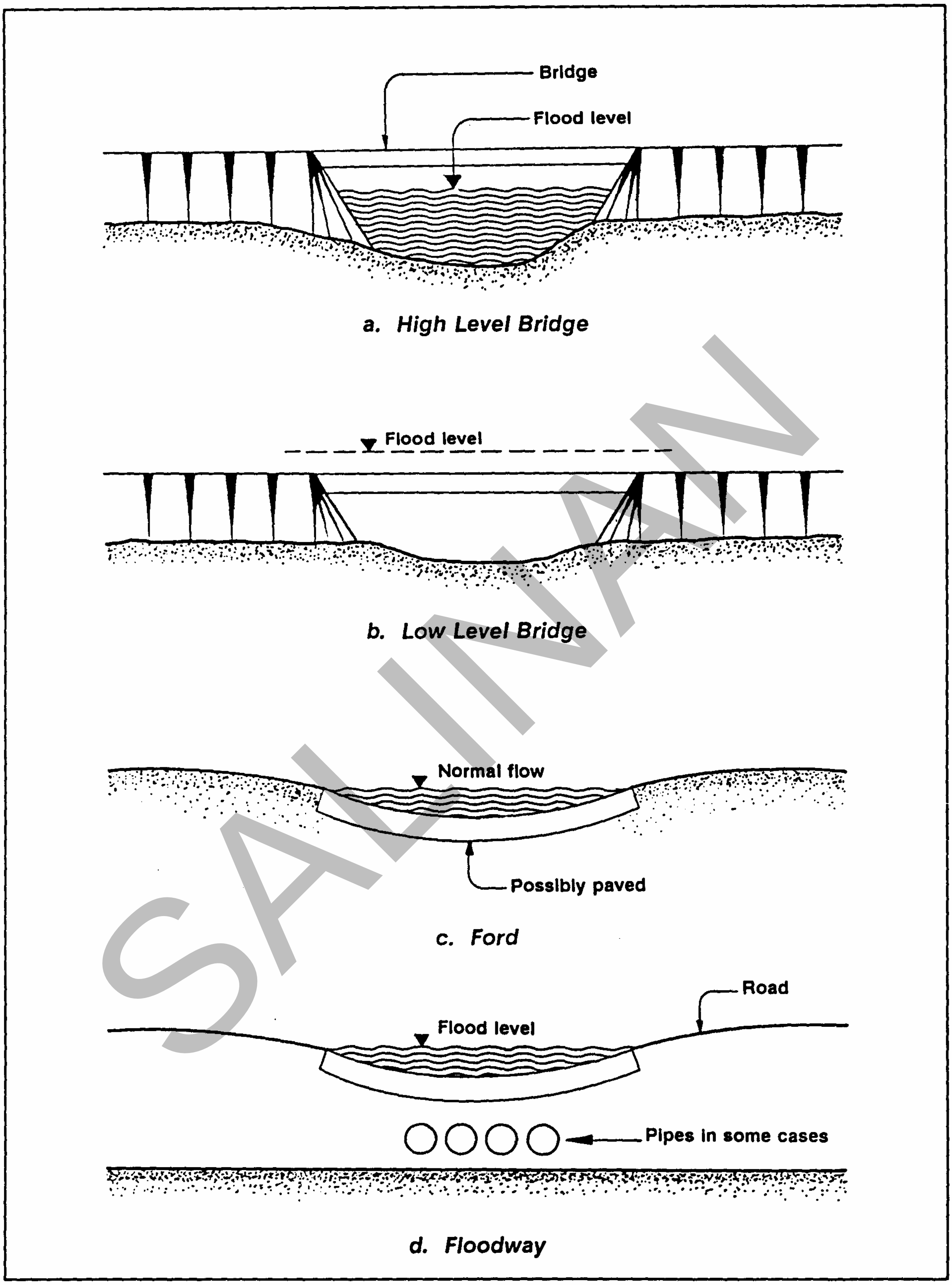

Figure 4.3 - General Types of Bridges

4.3.4 Combined Arrangements

Finally, it is important to mention that all these general types of bridges and culverts can be used in combination. In many instances, it may be far more economical to provide for a combination of structures rather than, say, a single high level bridge. A typical example is that of a flood plain situation where no well defined channel exists, and the whole area is flooded during the design flood. In such a case, two or more bridges (Figure 4.4a) or a bridge in conjunction with a floodway (Figure 4.4b) or a bridge with a battery of culverts (Figure 4.4c) could be used.

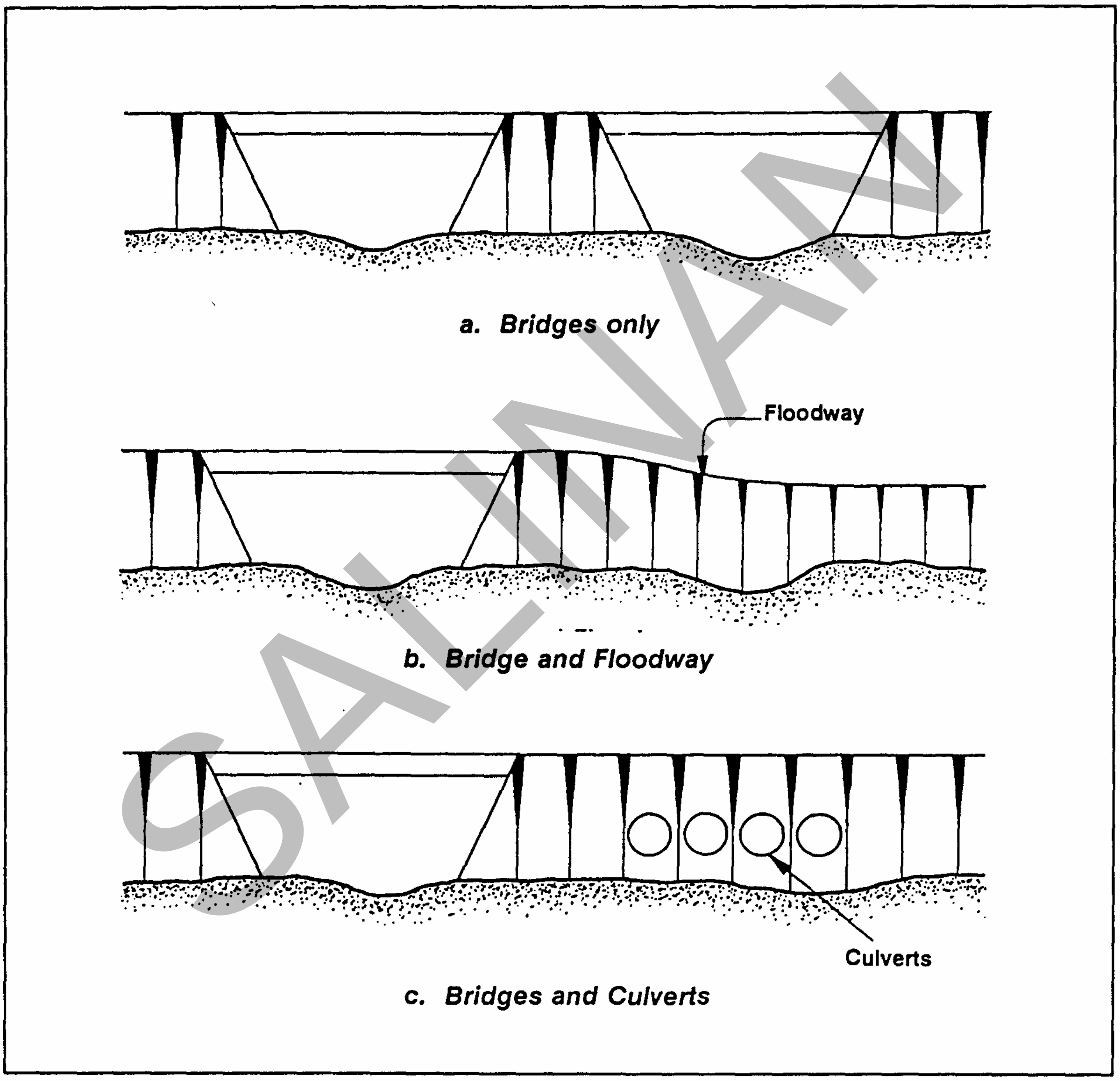

Figure 4.4 - Flood Plain Structures

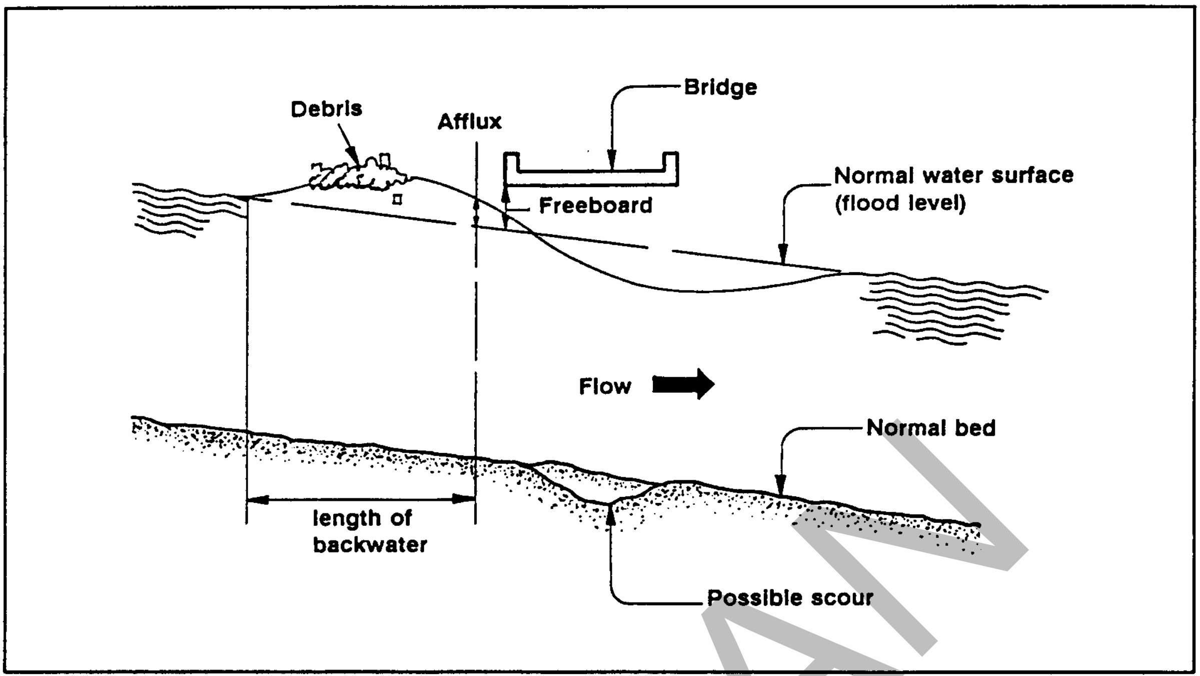

Figure 4.5 - Bridge during Flood

4.3.5 Waterway Calculations

At this stage, for each of the alternative alignments proposed, a good idea of the number and type of structure required has been obtained. In order to cost out each proposal as far as bridging is concerned, the length of the bridges to be used and the number and size of any culverts proposed must be obtained. This requires a hydrologic and hydraulic study for each possible site. In many instances, rough estimates only are made at this stage, the detailed investigation being left to the time when a definite crossing has been selected. However, it must be remembered that the more thorough the waterway calculations are now the less likely that there will be a major change later on.

In order to obtain an idea of the dimensions of the structures to be used for each proposal, the bridge designer must first be supplied with or decide upon the magnitude and frequency of the design flood. The frequency is usually dependent on the importance of the road and set by the design standards. The magnitude of the corresponding flood must, however, be estimated by various methods.

The design flood discharge then allows the designer to look at and calculate, as applicable, the following stream data :

- the design flood level
- the waterway area required
- the velocity through the structures
- the afflux or the head of water built up by the construction

- the presence and type of debris, and hence the amount of clearance or freeboard
- normal water levels
- navigation clearances, if applicable.

A basic understanding of what each of the above stream data means and how it affects the size of the structure is necessary. The meanings of most of the items listed can be best understood with reference to Figure 4.5. A high level bridge is considered.

The design flood level for the specified return period, for example, 50 year flood for normal structures, is usually taken to be the flood level for the unrestricted channel. This is usually calculated from the design discharge and site characteristics, or even from historical records.

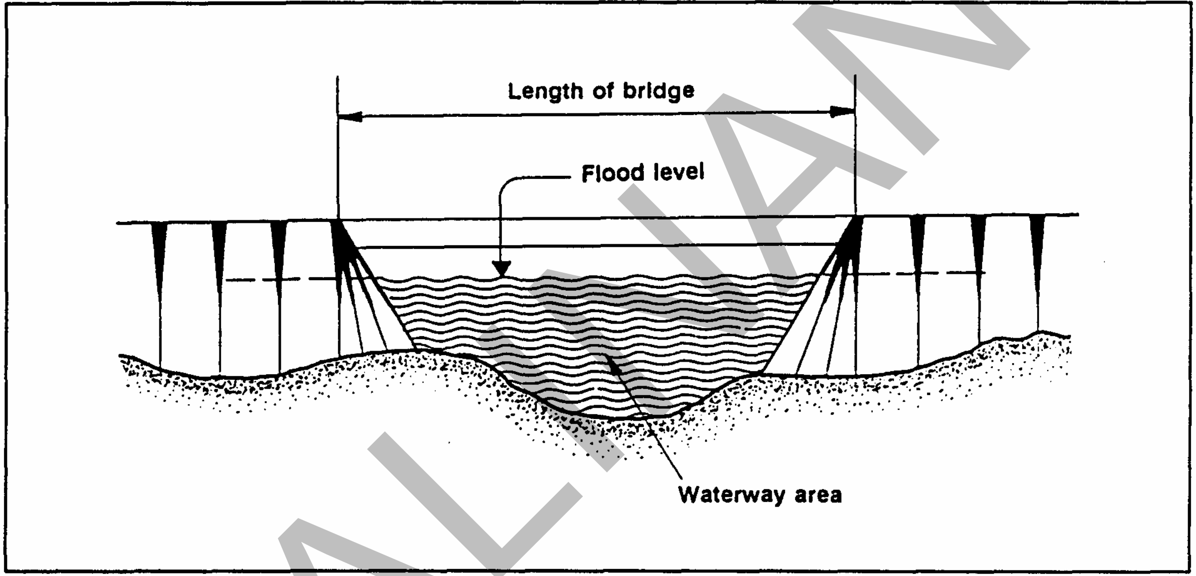

Figure 4.6 - Waterway Area

The amount of waterway to be provided determines the length of the bridge, and is basically defined as being the area below flood level at the site with the proposed bridge in place (see Figure 4.6).

The waterway area must be of sufficient amount so as to keep the velocity of flow through the structure within acceptable limits so that no or tolerable scouring occurs, and to maintain the backwater effect or afflux within specified limits. The afflux (see Figure 4.5) is the heading up which occurs upstream from the crossing as a consequence of introducing such a constriction in the stream. It becomes an important consideration if properties upstream of the bridge are likely to be flooded as a result of building the new structure. In many cases, it is also important to determine the extent of this heading up or backwater effect (see Figure 4.5).

Where a bridge is designed to pass floods of high return periods, clearance is provided between the underside of the structure and the design flood level, to provide for the passage of debris. This clearance is known as freeboard. The amount of freeboard, which also determines the level of the bridge deck, depends on the likely incidence and size of debris.

Normal water level as well as minimum water levels are useful for both designer and builder.

Other hydraulic considerations, including some of the criteria described above the high level bridge, must be taken into account for other types of bridges and for culverts. The hydraulic investigation can become very complex especially in cases where several structures, for example, a bridge and a battery of culverts, must be provided at the same crossing. The calculations used to be very approximate and ad-hoc, but nowadays with the availability of backwater curve computer packages a more rational approach can be applied.

4.4 SOIL INVESTIGATIONS

At this early stage of deciding on the best location to adopt, a very preliminary soil investigation should be carried out to determine the suitability or not of the various sites for the bridge types the designer wants to use.

Soil conditions can and do vary from site to site and this can affect the overall cost of the bridge. However, it is not normally worthwhile to spend a lot of time and money on soil testing. In the large majority of cases, for bridge structures, the choice will be between driven piles, spread footings or in-situ pile foundations. At this stage, the preliminary foundation investigation must be sufficient to allow a tentative judgement to be made of the foundation type suitable and allow comparative estimates to be made.

Some of the common methods used are :

- Inspection of the site to look at the general soil condition, for example, presence of rock, type of soil, etc.

- Look at information already available such as existing bridges at nearby locations, and geological maps.

- In some instances, test holes can be drilled but the timing of preliminary bores depends usually on the importance of the structure. And, this is only carried out after the site location has been chosen.

4.5 COSTS AND OTHER CONSIDERATIONS

The various possible crossings are basically compared on a cost basis. Therefore economy is of prime importance. Additional factors which must be looked at for each alternative include :

- Need and extent of land and building acquisition.

- Need to maintain a smooth traffic flow during construction. it is usually preferable to retain an existing bridge to carry this traffic rather than dismantle it to enable a new bridge to be constructed. Stage constructed in part widths is usually slow and expensive.

- A knowledge of the availability, quality and cost of construction materials should be obtained. The suitability of areas close to the crossing for setting up a construction depot, stock piles, a casting yard, etc. should be investigated. And, it is also important to find out if good access for vehicles carrying bridge materials, components and equipment is available.

4.6 FINAL SELECTION OF SITE

The final selection of bridge site is not as difficult as it might seem from the foregoing. Although it is rare for one alignment to satisfy all the requirements which must be considered, the bridge designer, to start off with, must be constrained to adopt the preferred line of the road designer. This is especially so in cases where the road costs far exceed the costs of bridging. It is also very rare that this preferred line is so poor from the point of view of bridge design and construction that no feasible bridge layout is possible.

In most cases, once the types of structure and their dimensions have been determined for each route it is easy to cost these alternatives and, in consultation with the road design engineer, make a recommendation. The general type of bridge to be used is almost invariably preset by the importance of the road, and preliminary waterway calculations will quickly give a good idea of the bridge dimensions. It is often only then that soil conditions and other requirements listed in Section 4.5 are considered.

4.7 REFERENCES

English Language References

Reference **Publication**

4.1 Tin Loi F., *Lecture Notes for Indonesian Bridge Engineering Course*, University of New South Wales, School of Civil Engineering, translated to Indonesian by the Civil Engineering Department, Bandung Institute of Technology, sponsored by Indonesian Australian Steel Bridge Project, 1985 (?).

4.2 Faraday R.V. & Charlton F.G., *Hydraulic Factors in Bridge Design*, Published by Hydraulics Research Station Limited, Wallingford, Oxfordshire, Produced by Thomas Telford Ltd, London, 1983.

4.3 Neill C.R. (Editor), *Guide to Bridge Hydraulics*, Published for Roads and Transport Association of Canada by University of Toronto Press, 1973.

4.4 Raina V.K., *Consultancy and Construction Agreements for Bridges, Including Field Investigations*, Tata McGraw-Hill, New Delhi, 1989.

4.5 Bindra S.P., *Principles and Practice of Bridge Engineering*, Dhanpat Rai & Sons, Delhi, 4th Revised Edition 1979, 1986 reprint.

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DIRECTORATE GENERAL OF HIGHWAYS
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BRIDGE INVESTIGATION MANUAL

SECTION 5

HYDROLOGY

FEBRUARY 1993

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5. HYDROLOGY

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5. HYDROLOGY

5.1 INTRODUCTION

This section of the manual details the hydrological aspects of bridge site investigation. The required design flood return periods are given for various bridge types and procedures for estimating flood discharges in the bridge waterway are detailed.

5.2 OBJECTIVE

Hydrologic analysis is a most important step prior to the hydraulic design of a bridge waterway. Such an analysis is necessary for determining the rate of flow, runoff or discharge that the bridge waterway will be required to accommodate. The design discharge is a hydraulic load on the bridge waterway and the bridge structure and the determination of its magnitude and duration is a very important design aspect.

The objective of a hydrologic analysis, therefore, is to determine :

- the flood discharge in the bridge waterway for the appropriate design flood return period along with
- the depth of water flow, and
- water velocity

5.3 DESIGN FLOODS RETURN PERIODS

The return period (or recurrence interval) of a flood is the average interval of time which that flood event will be equalled or exceeded. The reciprocal return period is the probability of exceedance of the flood in any year, that is, the 100 year return period flood is the flood which will occur on average once in 100 years and which will have a probability of 0.01 or 1 percent.

Table 5.1 lists the design flood return periods to be used for the design of waterways for bridges, culverts and floodways.

The choice of return period used in selecting the design flood is generally based upon cost-benefit studies, taking into consideration the desired level of service to traffic and the damage that might result from the design flood being exceeded, that is, the cost of delays to traffic and the cost of repairing flood damage is balanced against the cost of providing a higher standard in the first instance.

Table 5.1 - Design Flood Return Periods

Works Category	Crossing Type	Return Period
Special Works	large and important bridges	100 years
Normal Works	permanent bridges	50 years
	culverts	
Temporary Works	temporary bridges	20 years
	floodways	
	construction side tracks	

5.4 ESTIMATION OF DESIGN FLOODS

Estimates of design floods can be based upon either streamflow or rainfall records. The use of rainfall based techniques being second-best to the direct analysis of streamflow data. Unfortunately, in many countries including Indonesia, rainfall data is more readily available than streamflow data and most estimates of design floods have to be based upon it, with historic flood information being used to substantiate the results.

Ignoring the physical shape of a catchment, runoff will vary with rainfall, vegetation, soil type etc. It is obvious therefore, that methods for predicting runoff have to be derived or tested for each zone or region in which the hydrology is reasonably homogeneous.

The methods that can be used to estimate *design flood flows* can be separated into two broad groups as follows :

- **Streamflow-Based Methods**

  For gauged catchments with sufficient length of record (generally at least 15 years are required), the data can be statistically analysed and estimates made of design flows with particular recurrence intervals. The analysis of the historic flood frequency is the most reliable method for estimating the magnitude and frequency of future floods.

  Where there are a number of catchments in a region with sufficient length of record, the data can be analysed and the design flows related to catchment characteristics (for example, area, mainstream length, etc). These relationships can then be used to estimate design flows in ungauged catchments in the area. This approach is known as a *regional flood frequency method*.

- **Rainfall-Based Methods**

  For gauged catchments which have insufficient length of record to carry out a flood frequency analysis, the available flow data and pluviograph data can be used to obtain the parameters of a model of the catchment (that is, *unit hydrograph* or *runoff routing model*). A design storm can then be applied to the resulting model to give the required design flood.

  Where there are a number of catchments in an area with sufficient data to obtain the model parameters, these can be related to catchment characteristics to give a *synthetic unit hydrograph* (SUH) or *runoff routing procedure* for that area. This relationship can be used to obtain a model of ungauged catchments in the area to which design rainfall can be applied to obtain design flows.

  In area where streamflow and associated rainfall data are very limited, relationships between model parameters and catchment characteristics which have been obtained outside the area of interest can be tested on the data available, and the one which most closely models the catchment used.

The detailed procedures for estimating waterway discharges in Indonesia using either of the above methods for the required design flood return periods are given in Reference 5.1, *Banjir Recana untuk Bangunan Air*, disusun oleh Ir. Joesron Loebis, Departemen Pekerjaan Umum, Puslitbang Pengairan, Balai Penyelidikan Hidrologi, Bandung. This book is readily available through the DPU bookshop, Jl. Pattimurra 20, Kebayoran Baru, Jakarta.

Two personal computer programs have kindly been supplied by Puslitbang Pengairan, Balai Penyelidikan Hidrologi, Bandung, to assist in analysing hydrological data required to determine the waterway discharge.

These programs are :

- stream discharge frequency analysis, and
- rainfall intensity-duration curves.

The method of analysis, Fortran program listing, and procedure for deriving the discharge for the required *design flood return periods* using these programs is described in Reference 5.1.

These programs may be obtained from BIPRAN, Jl. Pattimurra 20, Kebayoran Baru, Jakarta, or directly from Puslitbang Pengairan, Balai Penyelidikan Hidrologi, Jl. Ir. H. Juanda 193, Bandung.

Refer to *Bridge Design Code*, Section 1.4.5, for other requirements in the estimation of design floods.

5.5 DESIGN FLOOD WATER LEVELS

Once the *design flood* peak discharge has been determined the water level and flow velocity in the stream can be calculated using the procedures detailed in Section 6, *Hydraulics*, of this *Manual*.

5.6 DESIGN FLOOD VERTICAL CLEARANCE

The vertical clearance between the lowest point of the bridge soffit and the *design flood* high water level shall be at least 1.0 metres. This clearance shall be increased if large-sized debris is likely. (Refer to *Bridge Design Code*, Section 1.4.4).

5.7 REFERENCES

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DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 6

HYDRAULICS

FEBRUARY 1993

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6. HYDRAULICS

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6. HYDRAULICS

6.1 INTRODUCTION

This section of the manual outlines the principles of flow in open channels as a background to waterway design. The design of bridge and culvert waterways is also detailed including methods of computing waterway discharges, backwater curves and flow behaviour for typical geometric arrangements.

6.2 OPEN CHANNEL FLOW

6.2.1 Types of Flow

a. General

Flow in open channels is classified as *steady flow* or *unsteady flow*. The flow is said to be steady when the rate of discharge does not vary with time.

Steady flow is further classified as *uniform* when the channel cross-section, roughness, and slope are constant, and as *non-uniform* or varied when the channel properties vary from section to section.

Depth of flow and mean velocity will be constant for steady flow in a uniform channel.

b. Uniform Flow

With a given depth of flow d in a uniform channel, the mean velocity V (m/s) may be calculated using the *Manning equation* :

$$V = \frac{R^{2/3}\, S^{1/2}}{n} \qquad (6.1)$$

Where V = mean velocity of flow (m/sec)

R = hydraulic radius = A/P

A = area of cross-section of flow (m^2)

P = wetted perimeter of cross-section of flow (m)

S = slope (m/m)

n = Manning roughness coefficient

The discharge Q (m^3/s) is then

$$Q = A\,V \qquad (6.2)$$

The *Manning equation* will give a reliable estimate of velocity, only if the discharge, channel cross-section, roughness and slope are constant over a sufficient distance to establish uniform flow conditions. Strictly speaking, uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes however, the Manning equation can be applied to most stream flow problems by making judicious assumptions. When the requirements for uniform flow are met, the depth ***d*** and velocity ***V*** are said to be normal and the slopes of the water surface and the channel bed are parallel. For practical purposes minor undulations in the stream bed or minor deviations from the mean cross-section can be ignored as long as the mean slope of the channel can be represented as a straight line.

c. Energy of Flow

Flowing water contains energy in two forms, *potential* and *kinetic*.

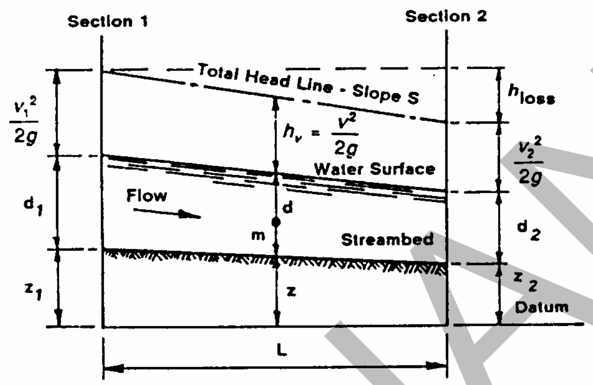

Figure 6.1 - Characteristics of Open Channel Flow

The potential (or latent) energy at a particular point is represented by the depth of the water plus the elevation ***Z*** of the channel bed above a convenient datum.

The kinetic (or motive) energy, in metres is represented by the velocity head $V^2/(2g)$.

In channel flow problems it is often desirable to consider the energy content with respect to the channel bottom. This is called the specific energy or specific head ***E*** and is equal to the depth of water ***d*** plus the velocity head :

$$E = d + \frac{V^2}{2g} \tag{6.3}$$

At other times it is desirable to use the total energy (total head), which is the specific head plus the elevation of the channel bed above a selected datum. For example, total head may be used in applying the energy equation, which states that the total head at one point in a channel carrying a flow of water is equal to the total head at any point downstream plus the energy (head) losses occurring between the two points. The energy (*Bernoulli*) equation is usually written :

$$d_1 + \frac{V_1^2}{2g} + Z_1 = d_2 + \frac{V_2^2}{2g} + Z_2 + h_{loss} \tag{6.4}$$

Note that in Figure 6.1 the line obtained by plotting velocity head above the water surface is the same line as that obtained by plotting specific head above the channel bed. This line represents the total energy, potential and kinetic, of the flow in the channel and is called the *total head line* or *total energy line*.

The slope ***S*** of the energy line is a measure of the friction slope or rate of energy head loss due to friction. The total head loss in length ***L*** is equal to ***S.L***. Under uniform flow conditions the energy line is parallel to the water surface and to the stream bed.

d. Critical Flow

The relative values of the potential energy (*depth*) and kinetic energy (*velocity head*) are important in the analysis of open channel flow. Consider, for example, the relation of the specific head, $d + V^2/(2g)$ and the depth d of a given discharge in a given channel at various slopes. Plotting values of specific head as ordinates and of the corresponding depth as abscissa will result in a specific head curve such as that shown in Figure 6.2.

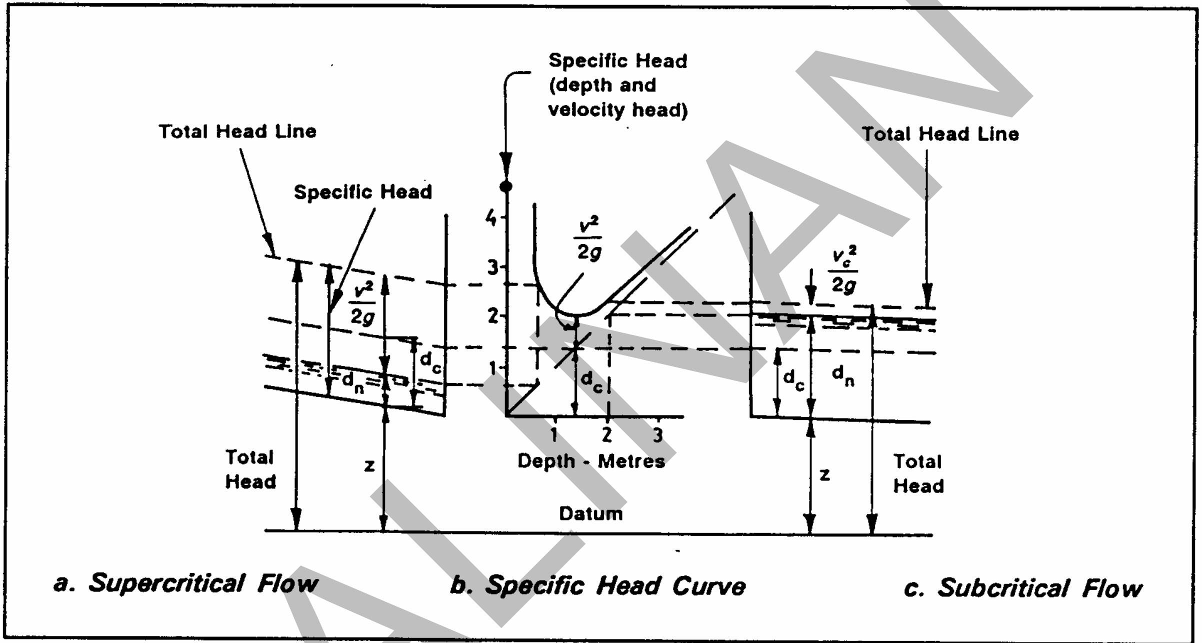

Figure 6.2 - Definition Sketch of Specific Head

The straight, diagonal line is drawn through points where depth and specific head are equal. This line thus represents the potential energy, and the ordinate interval between this line and the specific head curve is the velocity head for the particular depth. A change in the discharge ***Q*** or in the channel size or shape will change the position of the curve, but its general shape and location above and to the left of the diagonal line will remain the same. Note that the ordinate at any point on the specific head curve represents the total specific energy, $d + V^2/(2g)$ at that point. The lowest point of the curve represents flow with minimum energy. The depth at this point is known as critical depth d_c, and the corresponding velocity is the critical velocity, V_c. With uniform flow, the channel slope at which critical depth occurs is known as the critical slope S_c.

Points on the left of the low point of the specific head curve Figure 6.2 are for channel slopes steeper than critical and indicate relatively shallow depths and high velocities (Figure 6.2b). Such flow is called *supercritical flow*. This type of flow can occur in mountain streams. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually a point at which critical depth occurs.

Points on the right of the low point of the specific head curve (Figure 6.2b) are for slopes less than critical and indicate relatively large depths with low velocities (Figure 6.2c). Such flow is called subcritical flow. This type of flow occurs in streams in plains and broad valley regions.

In subcritical flow, the depth at any point is influenced by a downstream control, which may be either critical depth or the water surface in a lake or a large downstream channel.

The magnitude of critical depth depends only on the discharge and the shape of the channel, and is independent of the slope or channel roughness. Thus, for any given size and shape of channel, there is only one critical depth for a particular discharge.

Critical depth is an important value in hydraulic analysis because it is a control in reaches of non-uniform, flow, whenever flow changes from subcritical to supercritical. Typical situations in which critical flow occurs are :

- At a constriction, such as a culvert on a steep slope or with backwater.
- At the crest of a weir, such as a flood-crossing.
- At the outlet of a culvert discharging with a free outfall or into a relatively wide channel.

The potential and kinetic energy of flow in a channel can be expressed by the *Froude number*, defined as :

$$F = \frac{V}{\sqrt{gd}} \tag{6.5}$$

Where V = mean velocity of flow (m/s)

g = acceleration due to gravity (m/s^2)

d = hydraulic depth (m), which is defined as the cross-sectional area of the water normal to the direction of flow in the channel divided by the width of the free surface. For rectangular channels this is equal to the depth of the flow section.

When $F = 1$,

$$V_c = \sqrt{gd_c} \tag{6.6}$$

and the flow is said to be in a *critical state*.

If $F < 1$, or $V < \sqrt{(gd)}$, flow is *subcritical.*

If $F > 1$, or $V > \sqrt{(gd)}$, the flow is *supercritical.*

e. Non-Uniform Flow

Truly uniform flow rarely exists in either natural or man made channels, because changes in channel section, slope, or roughness cause the depths and average velocities of flow to vary from point to point along the channel, and the water surface will not be parallel to the stream bed. Flow which varies in depth and velocity along the channel is called non uniform.

Although flow in a generally uniform channel is not truly uniform, it is usually treated as uniform flow because uniform flow characteristics can readily be calculated, and the calculated values are usually close enough to the actual for all practical purposes.

With subcritical flow, a change in channel shape, slope, or roughness affects the flow for a considerable distance upstream, and thus the flow is said to be under downstream control. If a constriction such as a culvert causes ponding, or a bridge opening causes backwater, the water surface above the constriction will be a smooth curve asymptotic to the normal water surface upstream and to the water level at the pool or the bridge. This water surface profile is known as a backwater curve, and is characteristically very long.

Another example of downstream control occurs where an abrupt channel enlargement, as at the end of a culvert not flowing full, causes a drawdown in the flow profile to critical depth. The water surface profile upstream from a change in section or a break in channel slope will be asymptotic to the normal water surface upstream, but will drop away from the normal water surface on approaching the channel change. In this example, the flow is non uniform because of the changing water depth caused by changes in the channel section. Direct solution of open channel flow by the Manning equation is not possible in the vicinity of the changes in the channel section.

With supercritical flow, a change in channel shape, slope or roughness cannot be reflected upstream except for very short distances. However, the change may affect the depth of flow at downstream points; thus, the flow is said to be under upstream control.

6.2.2 Channel Rating

a. General

It is important that the normal stage height of a water course for a design flood discharge be determined as accurately as possible at the site of the stream crossing (that is, the bridge, culvert of flood-crossing). This may be accomplished from stream gauging records or, if these are unavailable, from a theoretical approach such as the slope area method, utilising records of peak floods as a check where these are available.

b. Slope Area Method

The following is a simplified variation of the slope area method utilising a single cross-section at the site of the stream crossing.

In streams of irregular cross-section, it is necessary to divide the water area for a particular stage height into smaller, but more or less regular subsections, assigning an appropriate retardance factor to each and calculating the discharge for each subsection separately, using the *Manning equation*. The total discharge can then be found by adding the discharges for each subsection. This can be repeated for other stage heights and a stage discharge rating curve drawn.

Care should be exercised in both the collection and use of field data, if errors are to be avoided in the final result.

c. Channel Roughness

A matter of prime importance in slope-area calculations is the ability to evaluate properly the roughness of the main channel and the flood plains, both are subject to extreme variations with vegetal growth and depth of flow. As a guide, values of the *Manning roughness coefficient* ***n***, as commonly encountered in practice, are tabulated for various conditions of channel and flood plain in Table 6.1 and 6.2. In interpreting roughness coefficients from Table 6.1, it should be kept in mind that the value of ***n***, for a small depth of flow, especially on a flood plain covered with grass, weeds, and brush, can be considerably larger than for greater flow depths over the same terrain. On the other hand, as the stage rises in a stream with an alluvial bed, sand waves develop which can increase the value of ***n***.

Table 6.1 - Manning Roughness Coefficient n for Minor Streams

MINOR STREAMS *Surface width at flood stage less than 30 m*		
Channel Type	**Channel Condition**	**Manning *n***
Relatively Regular Section	Some grass and weeds, little or no brush	0.030 - 0.035
	Dense growth of weeds, depth of flow materially greater than weed height	0.035 - 0.050
	Some weeds, light brush on banks	0.035 - 0.050
	Some weeds, heavy brush on banks	0.050 - 0.070
	For trees within channel, with branches submerged at high stage	*Increase above values by 0.010 - 0.020*
Irregular Section	Contains pools, slight channel meander	Increase above values by 0.010 - 0.020
Mountain Streams	No vegetation in channel, banks usually steep, trees and brush along banks, submerged at high stage	
	Bottom of gravel, cobbles and few boulders	0.040 - 0.050
	Bottom of cobbles, with large boulders	0.050 - 0.070

Table 6.2 - Manning Roughness Coefficient n for Flood Plains

FLOOD PLAINS		
Channel Type	**Channel Condition**	**Manning *n***
Pasture *no brush*	Short grass	0.030 - 0.035
	High grass	0.035 - 0.050
Cultivated Areas	No crop	0.030 - 0.040
	Mature row crops	0.035 - 0.045
	Mature field crops	0.040 - 0.050
Brush	Scattered brush, heavy weeds	0.050 - 0.070
	Light brush and trees	0.060 - 0.080
	Medium to dense bush	0.100 - 0.160
Trees	Clear land with tree stumps, no sprouts	0.040 - 0.050
	Clear land with tree stumps, with heavy growth of sprouts	0.060 - 0.080
	Heavy stand of timber, a few fallen trees, little undergrowth, flood stage below branches	0.100 - 0.120
	Heavy stand of timber, a few fallen trees, little undergrowth, flood stage reaching branches	0.120 - 0.160

Table 6.3 - Manning Roughness Coefficient n for Major Streams

MAJOR STREAMS Surface width at flood stage greater than 30 m		
Channel Type	**Channel Condition**	**Manning *n***
Regular Section	No boulders or brush	0.025 - 0.035
Irregular Section	Rough channel	0.035 - 0.100
*The Manning **n** is less than that for minor streams of similar description because banks offer less effective resistance.*		

Table 6.4 - Manning Roughness Coefficient n for Artificial Channels

ARTIFICIAL CHANNELS		
Channel Type	**Channel Condition**	**Manning *n***
Lined Channels	Concrete, smooth formed	0.012
	Bituminous concrete	0.013 - 0.016
Excavated Unlined Channels	Uniform section, short grass	0.022 - 0.027
	Relatively uniform section, grass, some weeds	0.025 - 0.030
	Relatively uniform section, dense weeds, deep channel	0.030 - 0.035
	Relatively uniform section, cobble bottom	0.030 - 0.040
Channels not Maintained *weeds and brush uncut*	Dense weeds as high as flow depth	0.080 - 0.120
	Clean bottom, brush on sides	0.050 - 0.080
	Dense brush, high stage of flow	0.100 - 0.140

6.3 BRIDGE WATERWAY DESIGN

6.3.1 Flow Characteristics

a. General

It is seldom economically feasible or necessary to bridge the entire width of a stream as it occurs at flood flow. Where conditions permit, approach embankments are extended out onto the flood plain to reduce costs, recognising that, in so doing, the embankments will constrict the flow of the stream during flood stages. This is an acceptable practice. When carried to extremes however, constriction of the flow can result in damage to bridges, costly maintenance, or even contribute to the complete loss of the bridge or the approach embankments.

The manner in which flow is contracted in passing through a channel constriction where the bed resists scour is illustrated in Figure 6.3. The flow bounded by each adjacent pair of streamlines is the same (25 m^3/s). Note that the channel constriction appears to produce practically no alteration in the shape of the streamlines near the centre of the channel. A very marked change is found near the abutments, however, since the momentum of the flow from the contracted portion of the channel must force the advancing central portion of the stream over to gain entry to the constriction. Upon leaving the constriction the flow gradually expands (5° to 7° per side) until normal conditions in the stream are again re-established.

Constriction of the flow produces loss of energy, the greater portion occurring in the expansion downstream. The loss of energy is reflected in a rise in the water surface and in the energy line upstream from the bridge. This is best illustrated by a profile along the centre of the stream, as shown in Figure 6.4a and 6.5a. The normal stage of the stream for a given discharge, before constricting the channel, is represented by the dash line labelled *normal water surface*. (Water surface is abbreviated as *WS* in the figures).

The nature of the water surface after constriction of the channel is represented by the solid line, *actual water surface*. Note that the water surface starts out above normal stage at Section 1, passes through the normal stage close to Section 2, reaches minimum depth in the vicinity of Section 3, and then returns to normal stage a considerable distance downstream, at Section 4. Determination of the rise in water surface at Section 1, denoted by the symbol h^*_1, and referred to as *bridge backwater*, is the primary objective of this section.

b. Types of Flow Encountered

There are three types of flow which may be encountered in bridge waterway design. These are labelled Types I, II and III on Figure 6.6. The long dash lines shown on each profile represent normal water surface, or the stage the design flow would assume prior to placing a constriction in the channel. The solid lines represent the configuration of the water surface on centreline of channel in each case, after the bridge is in place. The short dash lines represent critical depth, or critical stage in the main channel (y_{1c} and y_{4c}) and critical depth within the constriction, y_{2c}, for the design discharge in each case. Since normal depth is shown essentially the same in the four profiles, the discharge, boundary roughness and slope of channel must all increase in passing from Type I to Type IIA, to Type IIB, to Type III flow.

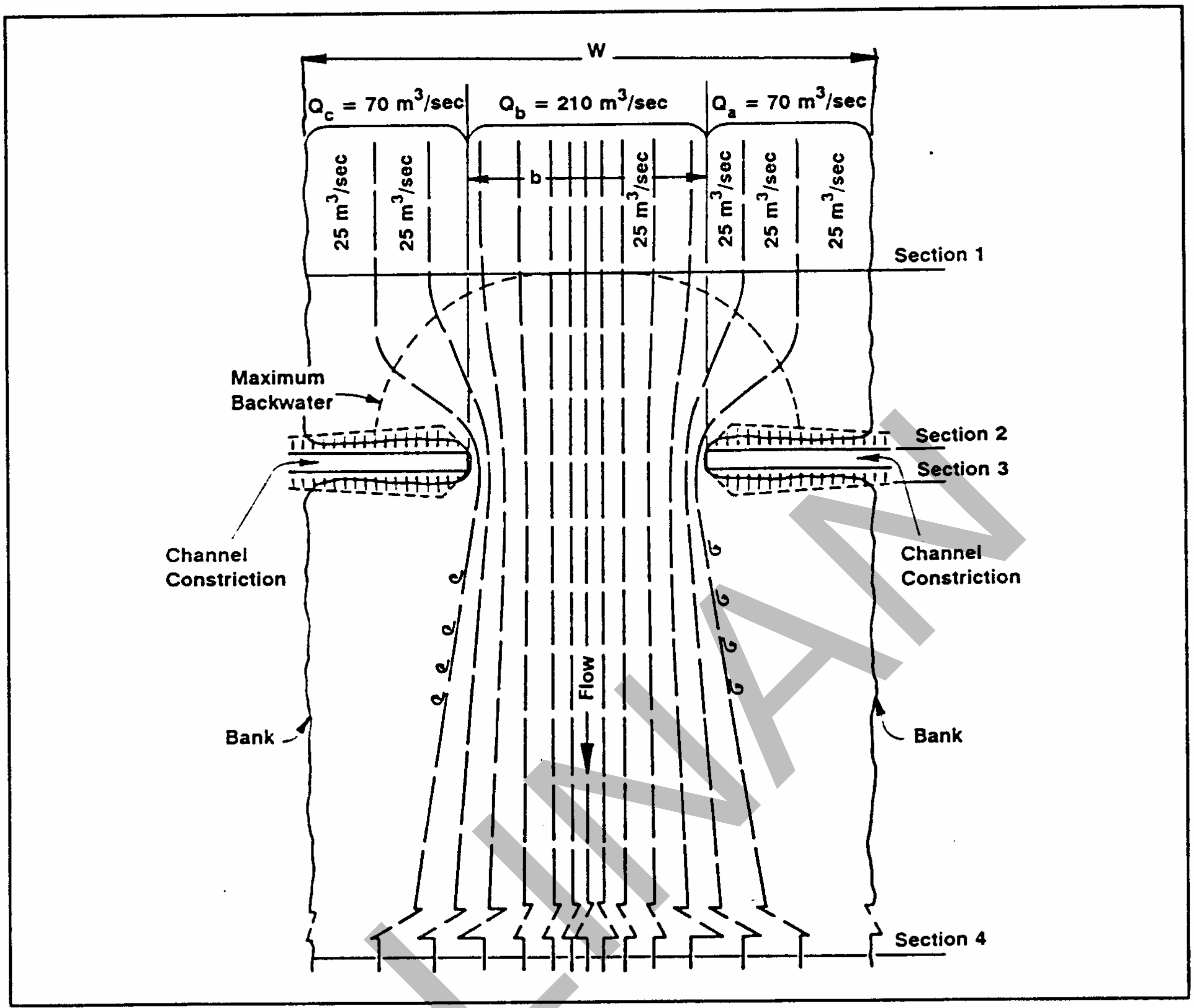

Figure 6.3 - Flow Lines for a Typical Normal Crossing

Type I Flow

Referring to Figure 6.6a, it can be observed that normal water surface is everywhere above critical depth. This has been labelled Type I or subcritical flow, the type usually encountered in practice. With the exception of Section 6.3.5, all design information in this section is limited to Type I, (subcritical flow). The backwater expression for Type I flow is obtained by applying the conservation of energy principle between Sections 1 and 4.

Type IIA Flow

There are at least two variations of Type II flow which will be described here under Types IIA and IIB. For Type IIA flow, Figure 6.6b, normal water surface in the unconstricted channel again remains above critical depth throughout but the water surface passes through critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at Section 4). Thus the backwater expression for Type I flow is not valid for Type II flow.

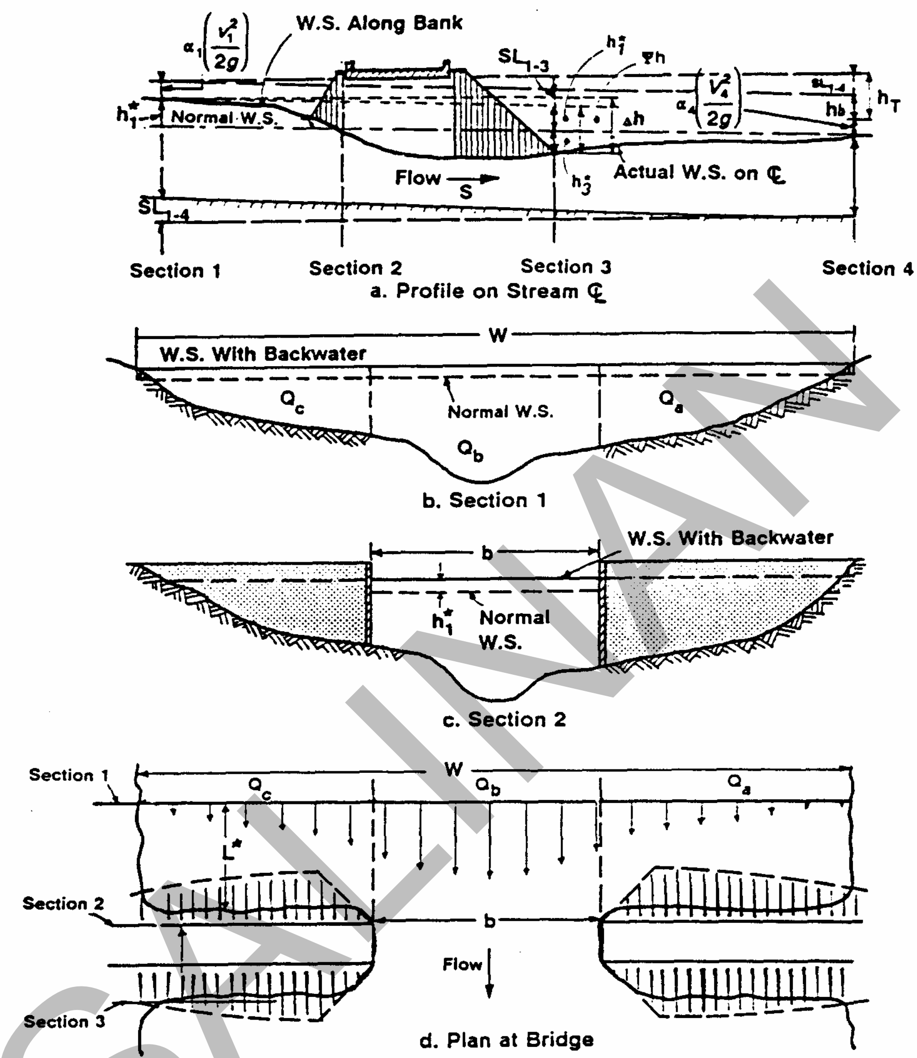

Figure 6.4 - Normal Crossing - Wingwall Abutments

Type IIB Flow

The water surface for Type IIB flow, Figure 6.6c, starts out above normal water surface and critical depth upstream, passes through critical depth in the constriction, next dips below critical depth downstream from the constriction and then returns to normal. The return to normal depth can be rather abrupt as in Figure 6.6c, taking place in the form of a weak hydraulic jump, since normal water surface in the stream is above critical depth. A backwater expression applicable to both Types IIA and IIB flow has been developed by equating the total energy between Section 1 and the point at which the water surface passes through critical

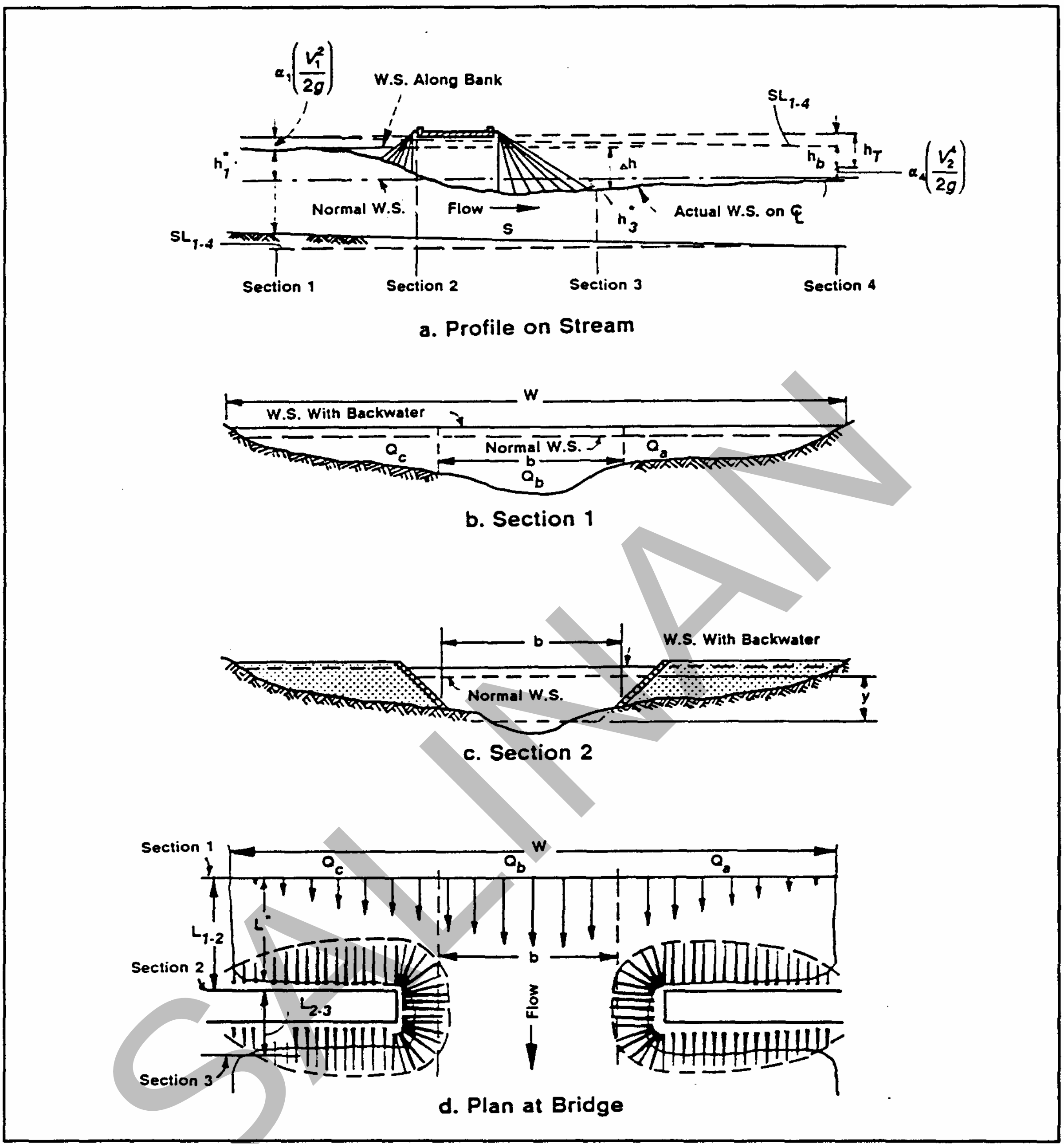

Figure 6.5 - Normal Crossing - Spillthrough Abutments

stage in the constriction.

Type III Flow

In Type III flow, Figure 6.6d, the normal water surface is everywhere below critical depth and the flow throughout is supercritical. This is an unusual case requiring a steep gradient but such conditions do exist, particularly in mountainous regions. Theoretically backwater should not occur for this type, since the flow throughout is supercritical. It is more than likely that an undulation of the water surface will occur in the vicinity of the constriction, however, as indicated on Figure 6.6d.

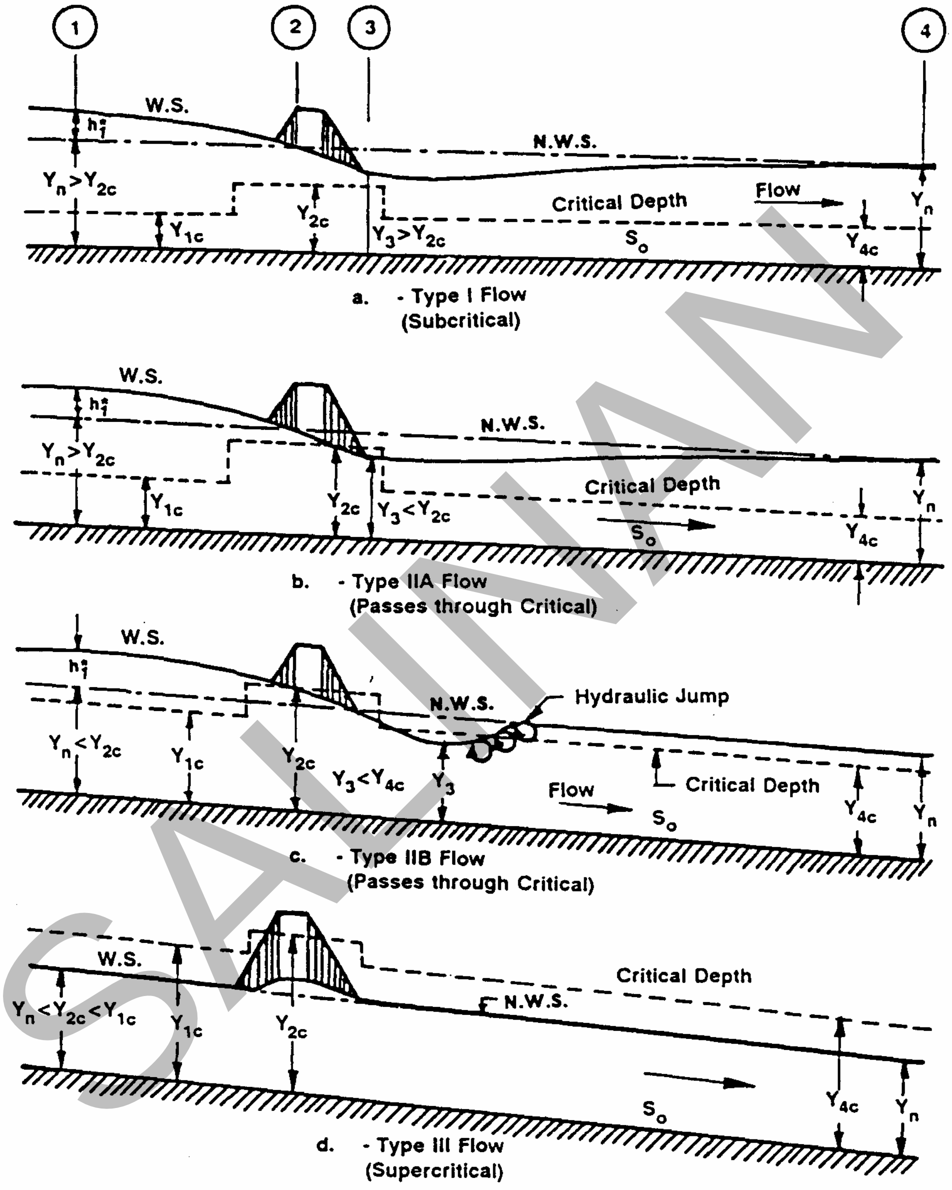

Figure 6.6 - Types of Flow Encountered

c. Definition of Symbols

Most of the symbols used in this section are recorded here for reference. Symbols not found here are defined where first mentioned.

A_1 = Area of flow including backwater at Section 1 (Figure 6.4b and 6.5b) (m^2).

A_{n1} = Area of flow below normal water surface at Section 1 (m^2).

A_{n2} = Gross area of flow in constriction below normal water surface at Section 2 (Figures 6.4c and 6.5c) (m^2).

A_4 = Area of flow at Section 4 at which normal water surface is re-established (Figure 6.4a) (m^2).

A_p = Projected area of piers normal to flow (between normal water surface and streambed) (m^2).

A_s = Area of scour measured on downstream side of bridge (m^2).

a = Area of flow in a subsection of approach channel (m^2).

b = Width of constriction (Figures 6.4c, 6.5c and Section 6.3.1 d.) (m).

b_s = Width of constriction of a skew crossing measured along centreline of roadway (Figure 6.11) (m).

C = h^*_{1s}/h^*_1 = Correction factor for backwater with scour.

C_b = Backwater coefficient for flow Type II.

e = Eccentricity $= (1-Q_c/Q_a)$ where $Q_c < Q_a$
or $= (1-Q_a/Q_c)$ where $Q_c > Q_a$

g = Acceleration due to gravity = 9.81 m/s^2

h^*_1 = Total backwater or rise above normal stage at Section 1 (Figs 6.4a and 6.5a) (m)

h^*_{1s} = Backwater with scour (m)

h^*_b = Backwater calculated from base curve (Figure 6.8) (m)

h_3 = Vertical distance from water surface on downstream side of embankment to normal water surface at Section 3 (Figure 6.4c and 6.5a) (m).

J = A_p/A_{n2} = ratio of area obstructed by piers to gross area of bridge waterway below normal water surface at Section 2 (Figure 6.9).

K_b = Backwater coefficient from base curve (Figure 6.8).

ΔK_p = Incremental backwater coefficient for piers (Figure 6.9)

ΔK_e	=	Incremental backwater coefficient for eccentricity (Figure 6.10).
ΔK_s	=	Incremental backwater coefficient for skew (Figure 6.12)
K^*	=	$K_b + \Delta K_p + \Delta K_e + \Delta K_s$ = total backwater coefficient for subcritical flow.
k	=	Conveyance in subsection of approach channel.
K_b	=	Conveyance of portion of channel within projected length of bridge at Section 1 (Figures 6.4b and 6.5b and Section 6.3.1 e.).
K_a, K_c	=	Conveyance of that portion of the natural flood plain obstructed by the roadway embankments (subscripts refer to left and right side, facing downstream (Figures 6.4b and 6.5b and Section 6.3.1 e.).
K_1	=	Total conveyance at Section 1 (Section 6.3.1 e.).
M	=	Bridge opening ratio (Section 6.3.1 f.).
n	=	Manning roughness coefficient (Section 6.3, Table 6.1 to 6.4).
p	=	Wetted perimeter of a subsection of a channel (m).
Q_b	=	Flow in portion of channel within projected length of bridge at Section 1 (Figure 6.3) (m^3/s).
Q_a, Q_c	=	Flow over that portion of the natural flood plain obstructed by the roadway embankments (Figure 6.3) (m^3/s).
Q	=	$Q_a + Q_b + Q_c$ = Total discharge (m^3/s).
r	=	a/p = Hydraulic radius of a subsection of flood plain or main channel (m).
S	=	Slope of channel bottom or normal water surface.
V_1	=	Q/A_1 = Average velocity at Section 1 (m/s).
V_4	=	Q/A_4 = Average velocity at Section 4 (m/s).
V_{n2}	=	Q/A_{n2} = Average velocity in constriction for flow at normal stage (m/s).
V_{2c}	=	Critical velocity in constriction for flow at normal stage (m/s).
w_p	=	Width of pier normal to direction of flow (Figure 6.9) (m).
W	=	Surface width of stream including flood plains (Figure 6.3) (m).
y_1	=	Depth of flow at Section 1 (m).
y_4	=	Depth of flow at Section 4 (m).
$\bar{y}$	=	A_{n2}/b = Mean depth of flow under bridge, referenced to normal stage, (Figure 6.5c) (m).

y_{1c} = Critical depth at Section 1 (m).

y_{2c} = Critical depth in constriction (m).

y_{4c} = Critical depth at Section 4 (m).

α_1 = Velocity head coefficient at Section 1 (Section 6.3.1 g.).

α_2 = Velocity head coefficient for constriction.

σ = Multiplication factor for influence of M on incremental back water coefficient for piers (Figure 7.9b).

ψh = $h^*_1 + h^*_3$ = for single bridge.

ϕ = Angle of skew (° degrees) (Figure 6.11).

d. Definition of Terms

Specific explanation is given below with respect to the concept of several of the terms and expressions frequently used throughout this section of the manual :

- **Normal Stage**

  Normal stage is the normal water surface elevation of a stream at a bridge site, for a particular discharge, prior to constricting the stream (see Figures 6.4a and 6.5a). The profile of the water surface is essentially parallel to the bed of the stream.

- **Normal Crossings**

  A normal crossing is one with alignment at approximately 90° to the general direction of flow during high water (as shown in Figure 6.3).

- **Eccentric Crossing**

  An eccentric crossing is one where the main channel and the bridge are not in the middle of the flood plain (Figure 6.10).

- **Skewed Crossing**

  A skewed crossing is one that is other than 90° to the general direction of the flow during flood stage (Figure 6.11).

- **Width of Constriction *b***

  No difficulty will be experienced in interpreting this dimension for abutments with vertical faces since ***b*** is simply the horizontal distance between abutment faces. In the more usual case involving spill through abutments, where the cross-section of the constriction is irregular, it is suggested that the irregular cross-section be converted to a regular trapezoid of equivalent area, as shown in Figure 6.5c. Then the length of bridge opening can be interpreted as :

e. Conveyance

$$b = \frac{A_{n2}}{\bar{y}} \tag{6.7}$$

Conveyance is a measure of the ability of a channel to transport flow. In streams of irregular cross-section, it is necessary to divide the water area into smaller but more or less regular subsections, assigning an appropriate roughness coefficient to each and calculating the discharge for each subsection separately. According to the *Manning formula* for open channel flow, the discharge in a subsection of a channel is :

$$q = \frac{a\, r^{2/3}\, S^{1/2}}{n} \tag{6.8}$$

By rearranging :

$$\frac{q}{S^{1/2}} = a\, r^{2/3} = k \tag{6.9}$$

where k is the conveyance of the subsection. Conveyance can, therefore, be expressed either in terms of flow factors or strictly geometric factors. In bridge waterway calculations, conveyance is used as a means of approximating the distribution of flow in the natural river channel upstream from a bridge. The method will be demonstrated in Section 6.3.6. Total conveyance K_1, is the summation of the individual conveyances comprising Section 1.

f. Bridge Opening Ratio

The bridge opening ratio, M, defines the degree of stream constriction involved, expressed as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the river. Referring to Figure 6.3 :

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q} \qquad \text{or} \quad M = \frac{210}{350} = 0.6 \tag{6.10}$$

The irregular cross-section common in natural streams and the variation in boundary roughness within any cross-section result in a variation in velocity across a river as indicated by the stream tubes in Figure 6.3. The bridge opening ratio, M, is most easily explained in terms of discharges, but it is usually determined from conveyance relations. Since conveyance is proportional to discharge, assuming all subsections to have the same slope, M can be expressed also as :

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K_1} \tag{6.11}$$

g. Kinetic Energy Coefficient

As the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along the banks, the average velocity head, calculated as $(Q/A_{n1})^2/(2g)$ for the stream at Section 1, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, α_1, defined as :

$$\alpha_1 = \frac{\sum (q\, v^2)}{Q\, V_{n1}^2} \tag{6.12}$$

where v = average velocity in a subsection.

q = discharge in same subsection.

Q = total discharge in river.

V_{n1} = average velocity in river at Section 1 or Q/A_{n1}

The methods of calculation will be further illustrated in Section 6.3.6.

A second coefficient, α_2, is required to correct the velocity head for nonuniform velocity distribution under the bridge :

$$\alpha_2 = \frac{\sum (q\, v^2)}{Q\, V_2^2} \tag{6.13}$$

where V_2 = average velocity in constriction = Q/A_2

The value of α_1 can be calculated but α_2 is not readily available for a proposed bridge. Figure 6.7 which relates α_2 to α_1 and the contraction ratio, M, is based upon actual measurements at bridge sites and may be used to estimate α_2. Because of the uncertainties involved in estimating α_2 it is suggested estimates of α_2 should err on the high side.

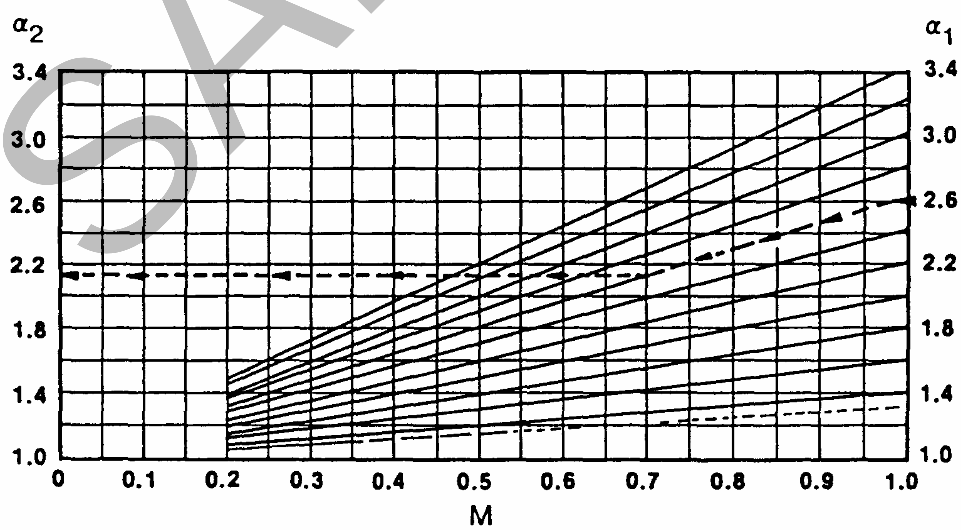

Figure 6.7 - Aid for Estimating α_2

6.3.2 Backwater

a. Expression for Backwater

This section presents a practical method for estimating the backwater caused by bridge constrictions.

The expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, Section 1, and a point downstream from the bridge at which normal stage has been re-established Section 4 (Figure 6.4a). The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross-sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between Sections 1 and 2, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

The expression for estimating backwater upstream from a bridge constricting flow is as follows :

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2 \right] \frac{V_{n2}^2}{2g} \qquad (6.14)$$

where	h_1	=	total backwater (m)
	K	=	total water coefficient
α_1 &	α_2	=	as defined in Equations (6.12) and (6.13) (Section 6.3.1 g.)
	A_{n2}	=	gross water area in constriction measured below normal stage (m^2)
	V_{n2}	=	average velocity in constriction or Q/A_{n2} (m/s)
	A_4	=	water area at Section 4 where normal stage is re-established (m^2)
	A_1	=	total water area at Section 1, including that produced by the backwater (m^2)

To estimate backwater, it is necessary to obtain the approximate value of h^*_1 by using the first part of Equation (6.14) :

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} \qquad (6.15)$$

The value of A_1 in the second part of Equation (4), which depends on h_1, can then be determined and the second term of the expression evaluated :

$$\alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \qquad (6.16)$$

b. Backwater Coefficient

Two symbols are interchangeably used throughout the text and both are backwater coefficients. The symbol K_b is the backwater coefficient for a bridge in which only the bridge opening ratio, M, is considered. This is known as a base coefficient and the curves on Figure 6.8 are called base curves. The value of the overall backwater coefficient, K^*, is likewise dependent on the value of M but also affected by :

- Number, size, shape, and orientation of piers in the constriction.
- Eccentricity or asymmetric position of bridge with respect to the valley cross-section, and
- Skew (bridge crosses stream at other than 90° angle).

It will be demonstrated that K^* consists of a base curve coefficient, K_b. to which is added incremental coefficients to account for the effect of piers, eccentricity and skew. The value of K^* is nevertheless primarily dependent on the degree of constriction of flow at a bridge.

c. Effect of *M* and Abutment Shape (Base Curves)

Figure 6.8 shows the base curves for backwater coefficient, K_b. plotted with respect to the opening ratio, M, for wingwall and spill through abutments. Note how the coefficient, K_b increases with channel constriction. The lower curve applies for 45° and 60° wingwall abutments and all spill through types. Curves are also included for 30° wingwall abutments and for 90° vertical wall abutments for bridges up to 60 m in length. These shapes can be identified from the sketches on Figure 6.8. Seldom are bridges with the latter type abutments more than 60 m long. For bridges exceeding 60 m in length, regardless of abutment type, the lower curve is recommended. This is because abutment geometry becomes less important to backwater as a bridge is lengthened. The base curve coefficients of Figure 6.8 apply to crossings normal to flood flow and do not include the effect produced by piers, eccentricity and skew.

d. Effect of Piers (Normal Crossings)

Backwater caused by introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p. which is added to the base curve coefficient K_b when piers are present in the waterway. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, has been assigned the letter J. In calculating the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure 6.9. By entering *Chart A* with the proper value of J and reading upward to the proper pier type, ΔK is read from the ordinate. Obtain the correction factor, σ, from *Chart B* for opening ratios other than unity. The incremental backwater coefficient is then :

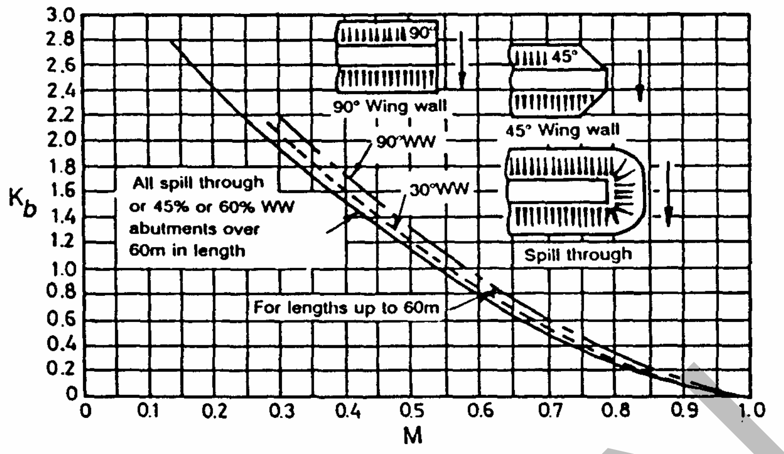

Figure 6.8 - Backwater Coefficient Base Curves (Subcritical Flow)

$$K_p = \sigma \, \Delta K \qquad (6.17)$$

The incremental backwater coefficients for pile bents can, for all practical purposes, be considered independent of diameter, width, or spacing of piles but should be increased if there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20% higher than that shown for bents with 5 piles. If there is a possibility of debris collecting on the piers, or piles, it is advisable to use a large value of J to compensate for the added obstruction. For a normal crossing with piers, the total backwater coefficient becomes :

$$K^* = K_b \ (\mathit{Figure}\ 6.8) + \Delta K_p \ (\mathit{Figure}\ 6.9) \qquad (6.18)$$

e. Effect of Piers (Skewed Crossings)

In the case of skewed crossings, the effect of piers is treated as explained for normal crossings (Section 6.3.2 d.) except for the calculation of J, A_{n2} and M. The pier area for a skewed crossing, A_p, is the sum of the individual pier areas normal to the general direction of flow, as illustrated by the sketch in Figure 6.9. Note how the width of pier W_p is measured when the pier is not parallel to the general direction of flow. the area of the constriction, A_{n2}, for skewed crossings is based on the projected length of bridge, $b_s \cos \phi$ (Figure 6.11). Again, A_{n2} is a gross value and includes the area occupied by piers. The value of J is the pier area, A_p, divided by the projected gross area of the bridge constriction, both measured normal to the general direction of flow. The calculation of M for skewed crossings is also based on the projected length of bridge, which will be further explained in Section 6.3.2 g. .

f. Effect of Eccentricity

Referring to the sketch in Figure 6.10, it can be seen that the symbols Q_a and Q_c at Section 1 are used to represent the portion of the discharge obstructed by the approach embankments. If the cross-section is extremely asymmetrical so that Q_a is less than 20% of Q_c or vice versa, the backwater coefficient will be somewhat larger than for comparable values of M shown on the base curve. The magnitude of the incremental backwater coefficient, ΔK_e, accounting for the effect of eccentricity, is shown in Figure 6.10. Eccentricity, e, is defined as 1 minus the ratio of the lesser to the greater discharge outside the projected length of the bridge, or :

$$e = 1 - \frac{Q_c}{Q_a} \quad \mathit{where}\ Q_c < Q_a \quad \mathit{or}$$
$$e = 1 - \frac{Q_a}{Q_c} \quad \mathit{where}\ Q_c > Q_a \qquad (6.19)$$

Reference to the sketch in Figure 6.10 will aid in clarifying the terminology. For instance, if $Q_c/Q_a = 0.05$, the eccentricity $e = (1-0.05)$ or 0.95 and the curve for $e = 0.95$ in Figure 6.10 would be used for obtaining ΔK_e. The largest influence on the backwater coefficient due to eccentricity will occur when a bridge is located adjacent to a bluff where a flood plain exists on only one side and the eccentricity is 1.0. The overall backwater coefficient for an extremely eccentric crossing with wingwall or spill through abutments and piers will be :

$$K^* = K_b \ (\mathit{Figure}\ 6.8) + \Delta K_p \ (\mathit{Figure}\ 6.9) + \Delta K_e \ (\mathit{Figure}\ 6.10) \qquad (6.20)$$

g. Effect of Skew

The method of calculation for skewed crossings differs from that of normal crossings as follows :

The bridge opening ratio, M, is calculated on the projected length of bridge rather than on the length along the centreline. The length is obtained by projecting the bridge opening upstream parallel to the general direction of flood flow as illustrated in Figure 6.11. The general direction of flow means the direction of flood flow as it existed prior to the placement of embankments in the stream. The length of the constricted opening is $b_s \cos\phi$ and the area A_{n2} is based on this length. The velocity head, $V_{n2}^2/(2g)$ to be substituted in Equation (6.14) is based on the projected area A_{n2}.

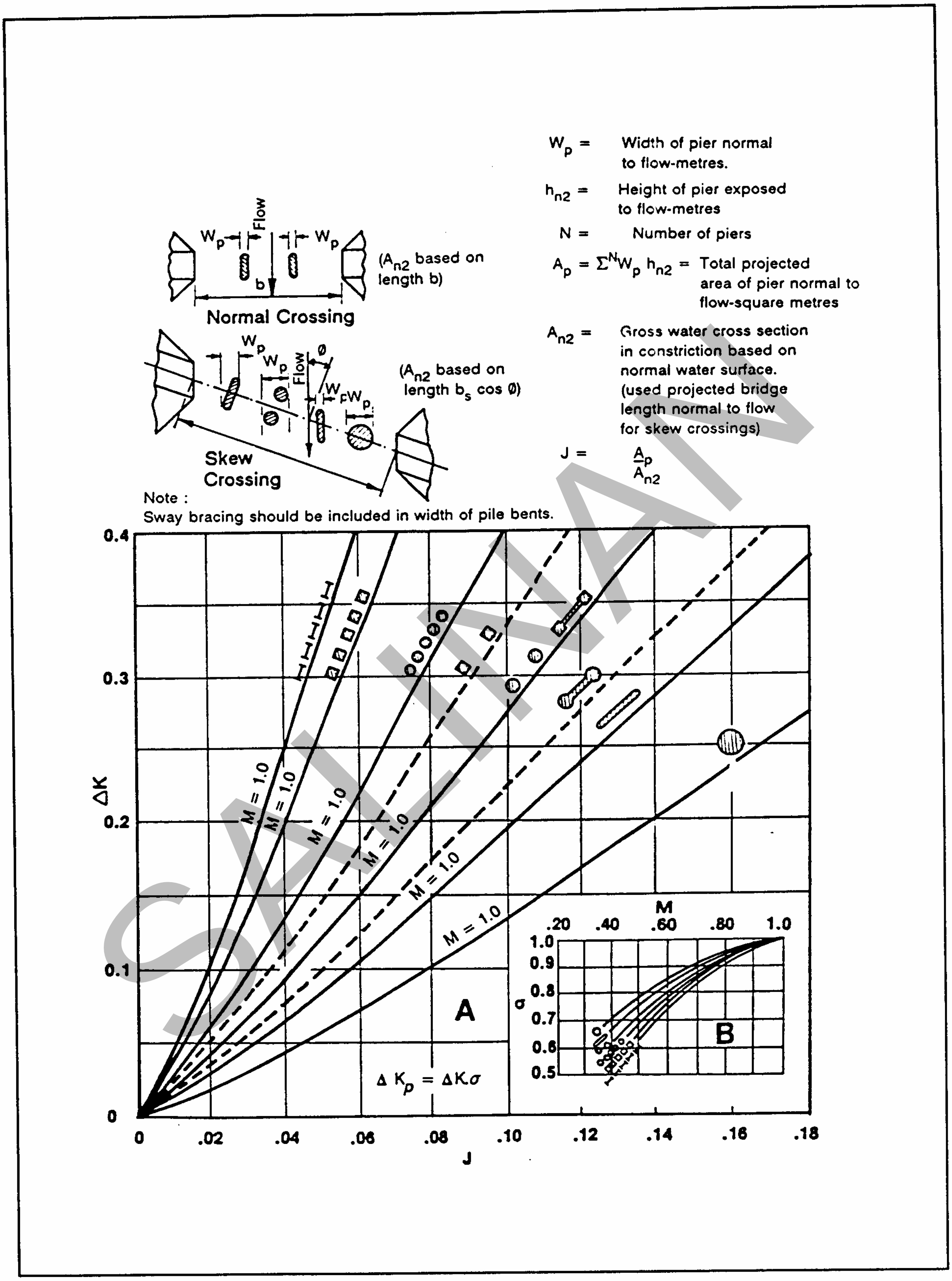

Figure 6.9 - Incremental Backwater Coefficient for Piers

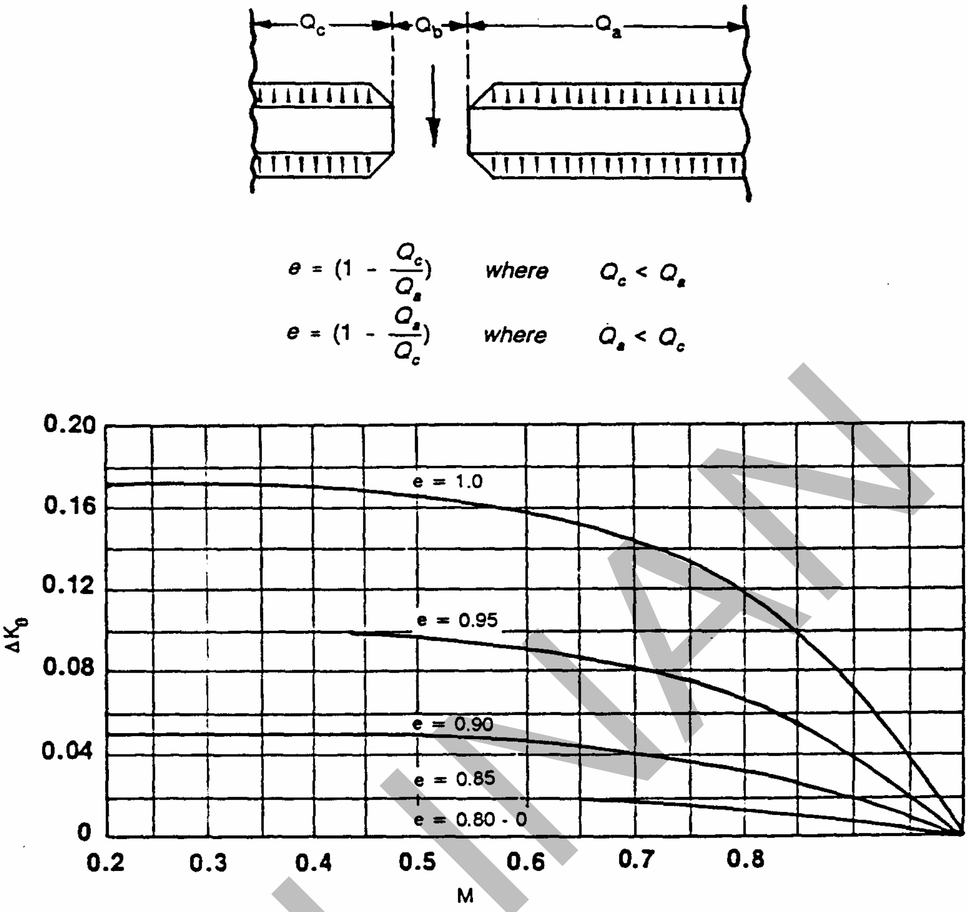

Figure 6.10 - Incremental Backwater Coefficient for Eccentricity

Figure 6.12 shows the incremental backwater coefficient, ΔK_s, for the effect of skew, for wingwall and spill through type abutments. The incremental coefficient varies with the opening ratio, ***M***, the angle of skew of the bridge ϕ, with the general direction of flood flow, and the alignment of the abutment faces, as indicated by the sketches in Figure 6.12. Note that the incremental backwater coefficient, ΔK_s, can be negative as well as positive. The negative values result from the method of calculation and do not necessarily indicate that backwater will be reduced by employing a skewed crossing. These incremental values are to be added algebraically to K_b obtained from the base curve. The total backwater coefficient for a skewed crossing with abutment faces aligned with the flow and piers would be :

$$K^{*} = K_b \text{ (Figure 6.8)} + \Delta K_p \text{ (Figure 6.9)} + \Delta K_s \text{ (Figure 6.12a)} \qquad (6.21)$$

Figure 6.13 was prepared using the same data used to construct Figure 6.12. By entering Figure 6.13 with the angle of skew and the projected value of ***M***, the ratio $b_s\cos\phi/b$ can be read from the ordinate. Knowing b and h^{*}_1 for a comparable normal crossing, one can solve for b_s, the length of opening needed for a skewed bridge to produce the same amount of backwater for the design discharge. The chart is especially helpful for estimating and checking.

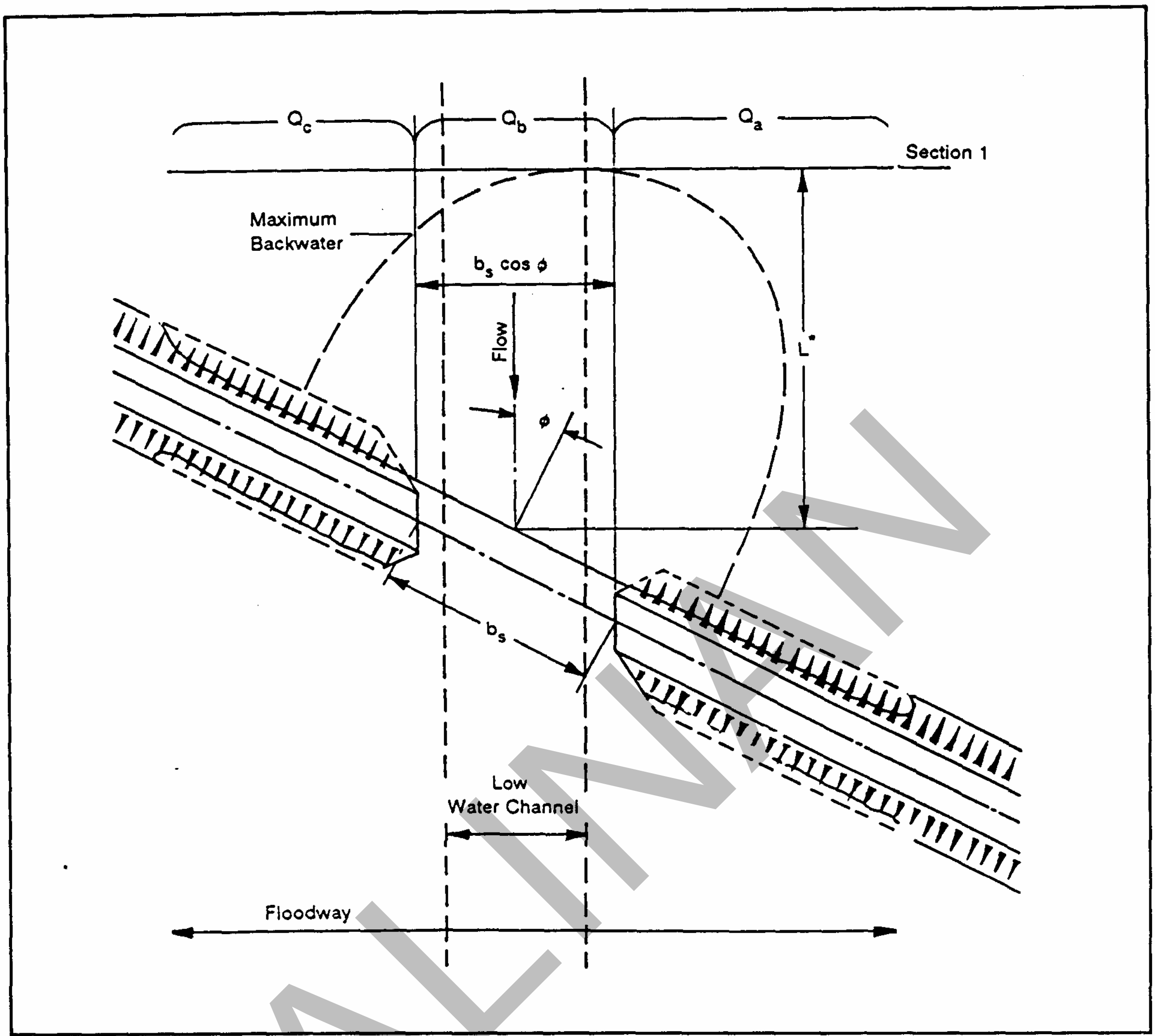

Figure 6.11 - Skewed Crossings

6.3.3 Effect of Scour on Backwater

a. General

The estimation of backwater in the preceding Section has been limited to the case where scour has not occurred. In actual practice where embankments have constricted the flow causing backwater and higher velocities through the bridge opening, scour will occur where the streambed is composed of loose or soft material (for an explanation of the scour phenomena, see Section 7, *Scour Prediction*, and Section 8, *Scour Protection*). The extent of scour will depend upon both the bed material and the velocity of flow. If a flood persists for a sufficient period of time equilibrium conditions will eventually result from the increase in waterway area, resultant reduction in backwater and velocity, and reduced capacity of the flow to cause further scour.

Figure 6.14 shows the effect of scour on the bridge backwater.

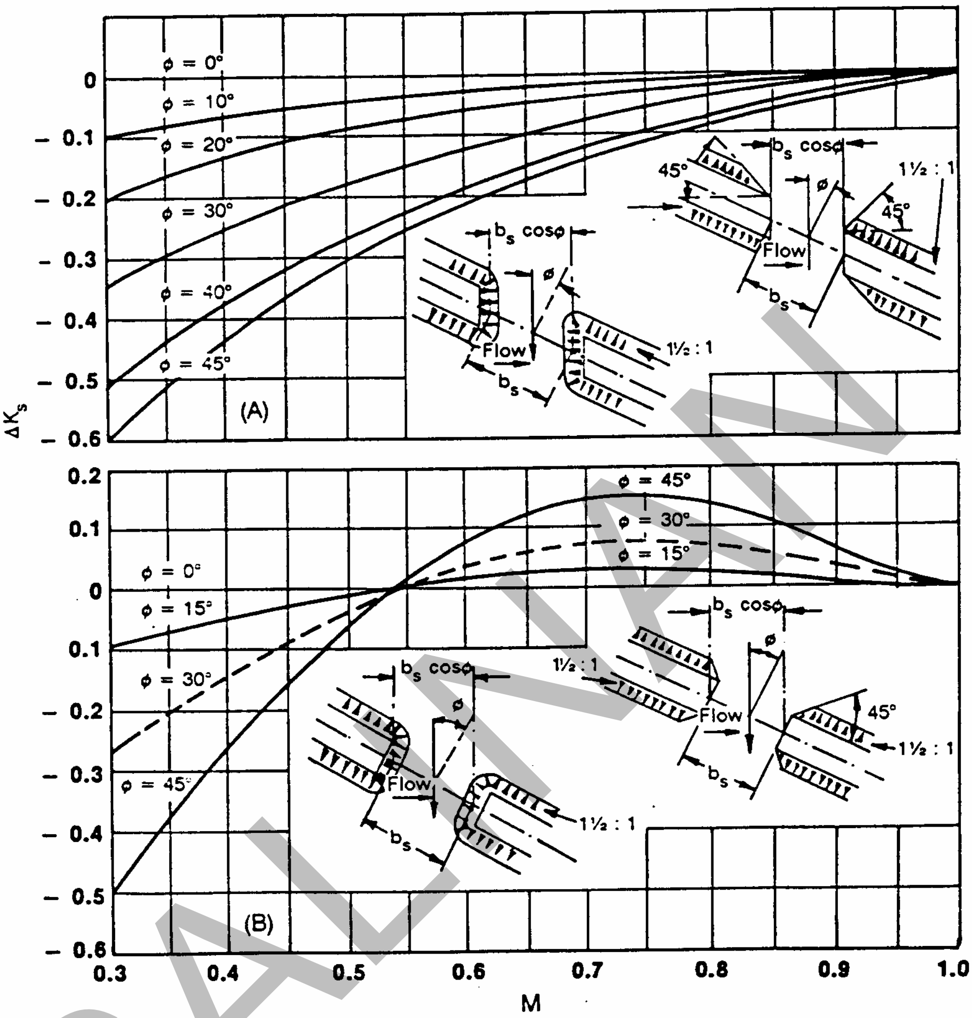

Figure 6.12 - Incremental Backwater Coefficient for Skew

In cases where the bridge foundations can be adequately protected (see Section 8) it may be advisable to encourage scour in the interest of utilising a shorter bridge. This same objective can be attained by enlarging the waterway area under a bridge with excavation machinery during construction. In such cases it is desirable to determine the amount of backwater to be expected with an increase in the waterway area.

b. Backwater Determination

A design curve derived from model experiments is included as Figure 6.15. The correction factor for backwater with scour ($C = h^*_{1s}/h^*_1$) is plotted with respect to A_s/A_{n2}, where the

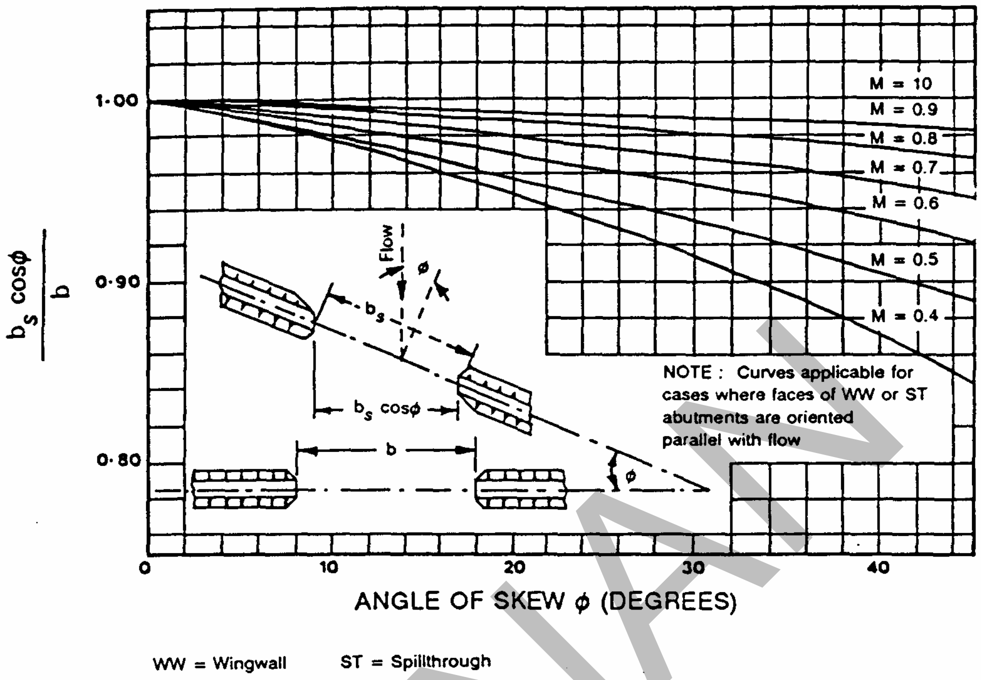

Figure 6.13 - Ratio of Projected to Normal Length of Bridge for Equivalent Backwater (Skewed Crossings)

terms bearing the subscript$_s$, designate values with scour; those not bearing this subscript represent the same values calculated with a rigid bed. Supposing the backwater at a given bridge was 0.5 m with no scour; it would be reduced to 0.26 m were scour to enlarge the waterway area by 50%, or it would be reduced to 0.16 m should the waterway area be doubled. The same reduction applies equally well to the ratios

$$\frac{h^*_{3s}}{h^*_3} \quad and \quad \frac{\psi h_s}{\psi h} \qquad (6.22)$$

so one curve will suffice for all three. Thus to obtain backwater and related information for bridge sites where scour is to be encouraged, where scour cannot be avoided, or where the waterway is to be enlarged during construction, it is first necessary to estimate the backwater and other quantities desired according to the method outlined in Section 6.3.2 for a rigid bed, using the original cross-section of the stream at the bridge site. These values are then multiplied by a common coefficient from Figure 6.14 as follows :

$$h_{1s}^{*} = C\, h_1^{*} \qquad (6.23)$$

$$h_{3s}^{*} = C\, h_3^{*} \qquad (6.24)$$

$$\psi h_s = C\, \psi h \qquad (6.25)$$

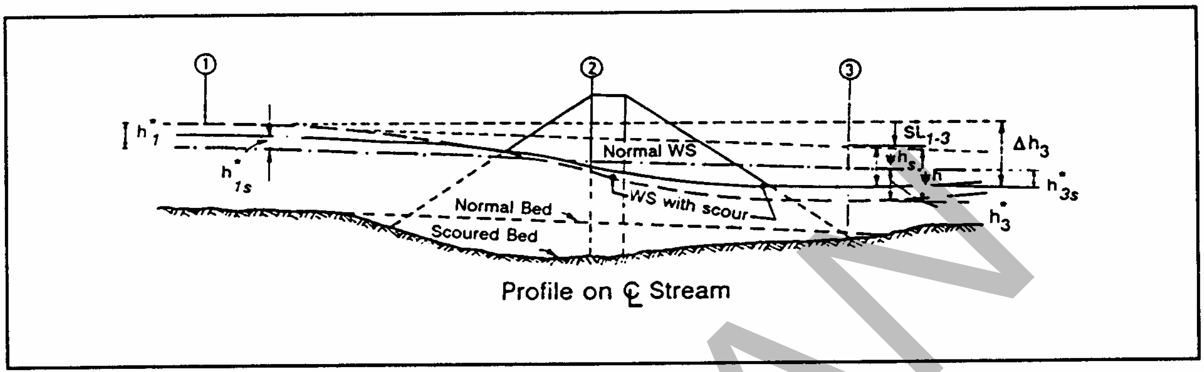

Figure 6.14 - Effect of Scour on Bridge Waterway

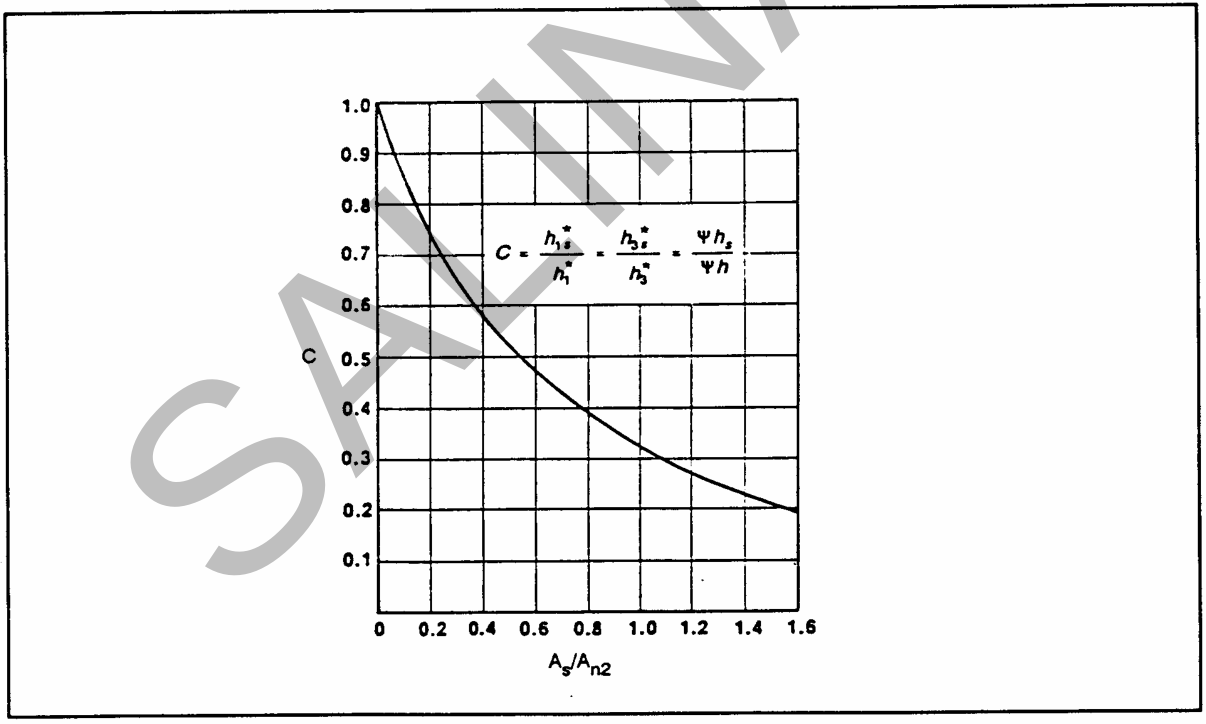

Figure 6.15 - Correction Factor for Backwater Scour

6.3.4 Superstructure Partially Inundated

a. General

Cases arise in which it is desirable to calculate the backwater upstream from a bridge or the discharge under a bridge when flow is in contact with the girders. Once flow contacts the upstream girder of a bridge, orifice flow is established so the discharge then varies as the square root of the effective head. The result is a rather rapid increase in discharge for a moderate rise in upstream stage. The greater discharge, of course, increases the likelihood of scour under the bridge. Inundation of the bridge deck is a condition the designer seldom contemplates in design but it occurs frequently on older bridges.

Two cases are considered below; the first where only the upstream girder is in the water as indicated by the sketch in Figure 6.16 and the second, where the bridge constriction is flowing full, all girders in the flow, as shown in Figure 6.17.

b. Case 1 - Upstream Girder in Flow

The most logical and simple method of approach is to treat this flow condition as a sluice gate problem (extreme case).

Using a common expression for sluice gate flow :

$$Q = c_d \, b_n \, Z \left[2g \left(y_u - \frac{Z}{2} + \alpha_1 \frac{V_1^2}{2g} \right) \right]^{\frac{1}{2}} \qquad (6.26)$$

where

where	Q	=	total discharge (m^3/s)
	c_d	=	coefficient of discharge
	b_n	=	net width of waterway - excluding piers (m)
	Z	=	vertical distance - bottom of upstream girder to mean river bed under bridge (m)
	y_u	=	vertical distance - upstream water surface to mean river bed at bridge (m)

For Case 1, the coefficient of discharge c_d, is plotted with respect to the parameter y_u/Z on Figure 6.16. The upper curve applies to the coefficient of discharge where only the upstream girder is in contact with the flow. By substituting values in Equation (6.26), it is possible to solve for either the water surface upstream or the discharge under the bridge, depending on the quantities known. It appears that the coefficient curve (Figure 6.16) approaches zero as y_u/Z becomes unity. This is not the case since the limiting value of y_u/Z for which Equation (6.26) applies is not much less than *1.1*. There is a transition zone somewhere between $y_u/Z = 1.0$ and *1.1* where free surface flow changes to orifice flow or vice versa. The type of flow within this range is unpredictable. For $y_u/Z = 1.0$, the flow is dependent on the natural slope of the stream, while this factor is of little concern after orifice flow is established or $y_u/Z > 1.1$.

In calculating a general river backwater curve across the bridge shown on Figure 6.16, it is necessary to know water surface elevation downstream as well as upstream from the bridge.

The approximate depth of flow, y_3, can be obtained from Figure 6.16 by entering the top scale with the proper value of y_u/Z and reading down to the upper curve, then over horizontally to the lower curve, and finally down to the lower scale as shown by the arrows. The lower scale gives the ratio of y_u/y_3.

c. Case 2 - All Girders in Contact with Flow

Where the entire area under the bridge is occupied by the flow, the calculation is handled in a different manner. To calculate the water surface upstream from the bridge, the water surface on the downstream side and the discharge must be known. Or if the discharge is desired, the drop in water surface across the roadway embankment, Δh, and the net area under the bridge is required. The experimental points on Figure 6.17, which are for both wingwall and spill through abutments, show the coefficient of discharge to be essentially constant at ***0.80*** for the range of conditions tested. The equation recommended for the average two to four lane concrete girder bridge for Case 2 is :

$$Q = 0.8\ b_n\ Z\,(2g\ \Delta h)^{1/2} \qquad (6.27)$$

where the symbols are defined as in Equation (6.26). Here the net width of waterway (excluding width of piers) is used again. It is preferable to measure Δh across embankments rather than at the bridge proper. The partially inundated bridge compares favourably with that of a submerged box culvert but on a large scale. Submergence, of course, can increase the likelihood of scour under a bridge.

For working up general backwater curves for a river it is desirable to know the drop in water level across the existing bridge as well as the actual water surface elevation either upstream or downstream from the bridge. Once Δh is calculated from Equation (6.27), the depth of flow upstream, y_u, can be obtained from *Chart B*, Figure 6.17, where $\bar{y}$ is depth from normal stage to mean river bed at the bridge in metres.

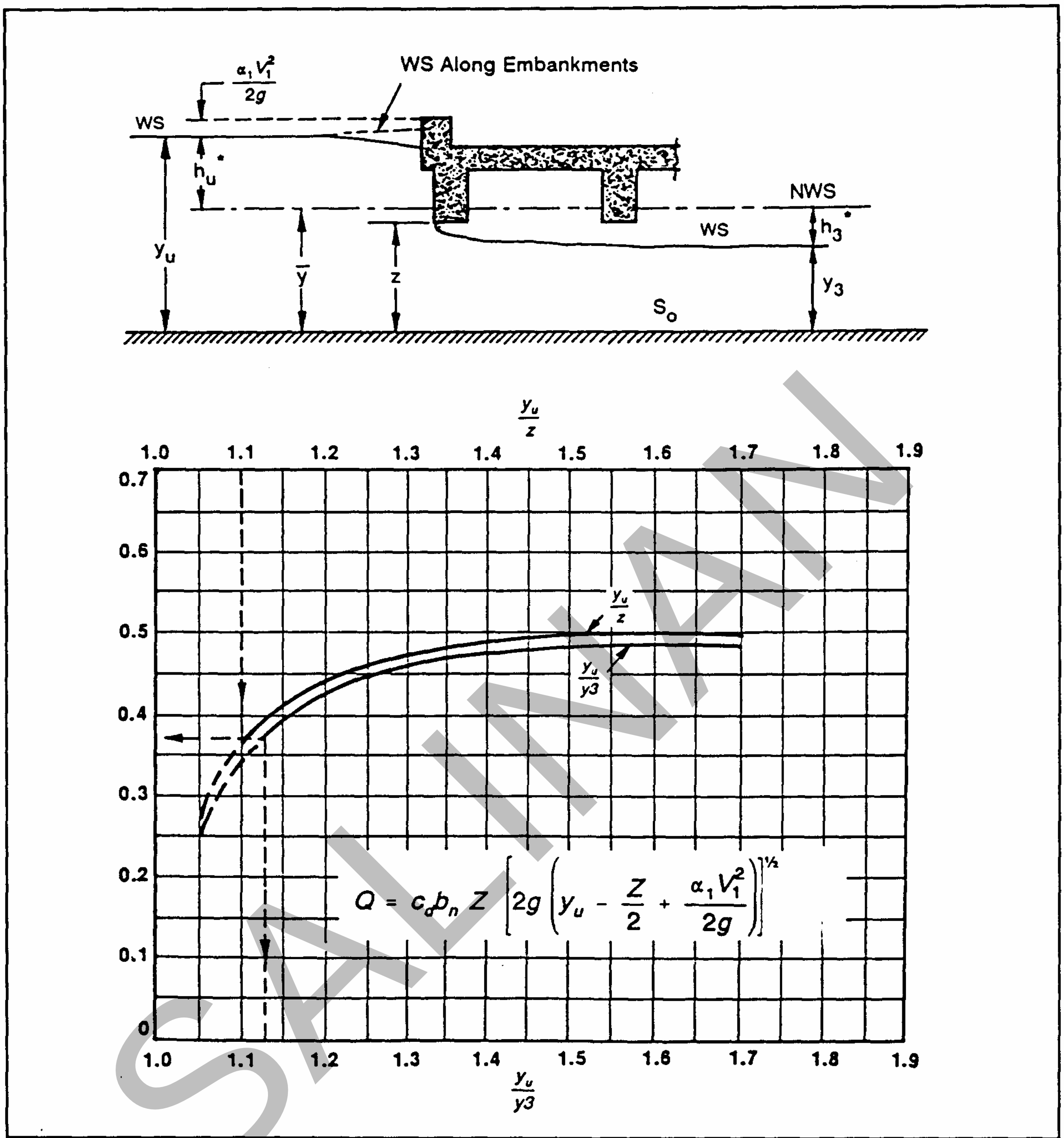

Figure 6.16 - Case 1 - Discharge Coefficients for Upstream Girder in Flow

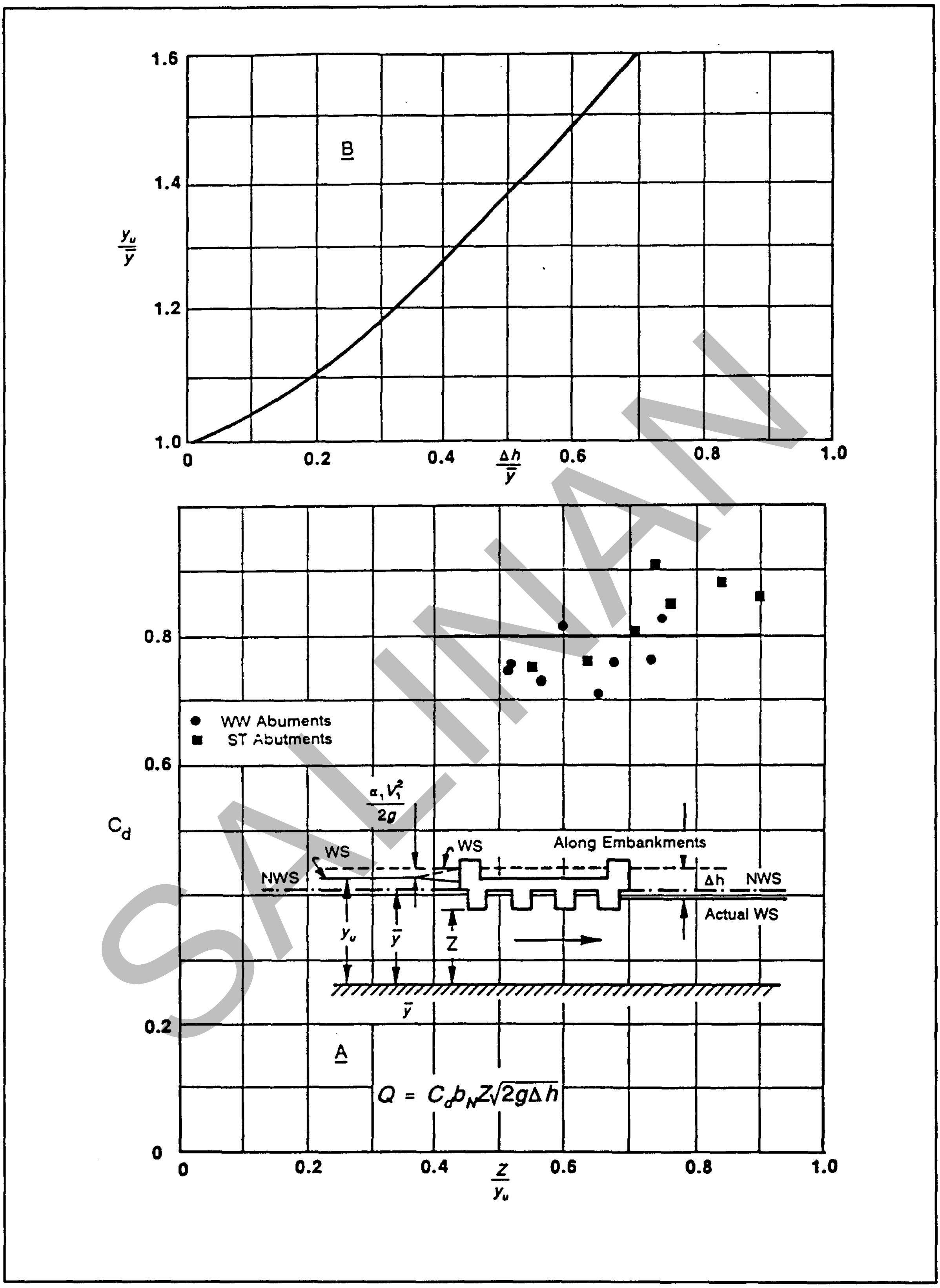

Figure 6.17 - Case 2 - Discharge Coefficient for all Girders in Flow

6.3.5 Flow Passes Through Critical Depth (Type II)

a. Introduction

The calculation of backwater for bridges on streams with fairly steep gradients, by the method outlined up to this point, may result in unrealistic values. When this occurs, it is probably a sign that the flow encountered is Type II (see Figure 6.6), and the backwater analysis for subcritical stage under the bridge but returns to normal or subcritical flow some distance downstream. In the case of Type IIB flow, the water surface passes through critical stage under the bridge and then dips below critical stage downstream. The sole source of data for Type II flow is from model studies, which cover but a limited range of contraction ratios.

b. Backwater Coefficients

The expression for the backwater coefficient for Type II flow is :

$$c_b = \frac{h_1^* + \bar{y} - y_{2c}}{\alpha_2 \dfrac{V_{2c}^2}{2g}} + \frac{\alpha_1}{\alpha_2}\left(\frac{V_1}{V_{2c}}\right)^2 - 1 \qquad (6.28)$$

where $\bar{y}$ = normal depth in constriction or A_{n2}/b (m)

y_{2c} = critical depth in constriction or A_{2c}/b (m)

V_{2c} = critical velocity in constriction or Q/A_{2c} (m/s)

A_{2c} = area in constriction below critical depth (m^2)

α_2 = velocity head coefficient for the constriction

The backwater coefficient has been assigned the symbol c_b to differentiate it from the coefficient for subcritical flow.

The curve of Figure 6.18 accounts for the contraction ratio only, which is the major factor involved. The effect of piers, eccentricity, and skew have not been evaluated because of the tentative nature of the curve. The incremental coefficients of Figures 6.9, 6.10, and 6.11 for piers, eccentricity and skew, are not applicable to Type II flow problems.

The backwater for Type II flow, with no allowance for piers, eccentricity and skew, is then :

$$h_1^* = \alpha_2 \frac{V_{2c}^2}{2g}(c_b+1) - \alpha_1 \frac{V_1^2}{2g} + y_{2c} - \bar{y} \qquad (6.29)$$

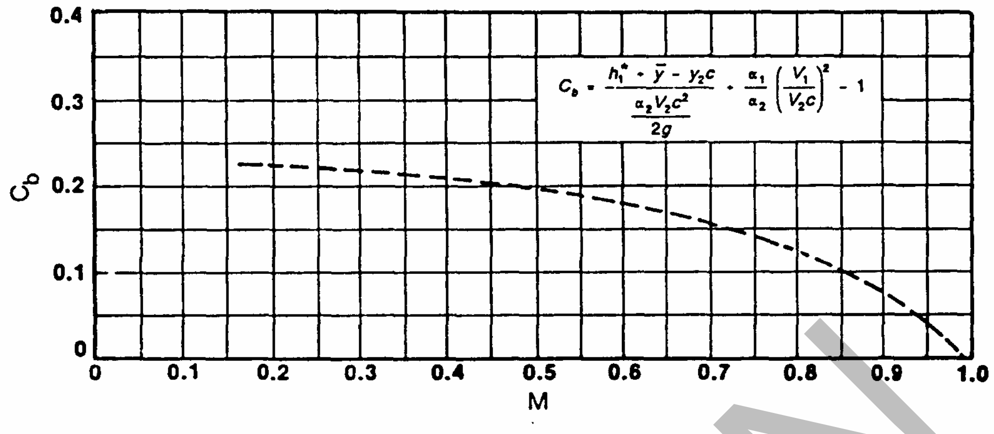

Figure 6.18 - Tentative Backwater Coefficient Curve for Type II Flow

c. Recognition of Flow Type

The prime difficulty here will be to determine which type of flow occurs at a proposed bridge site in the field prior to starting the backwater calculations. No definite answers can be given since most problems encountered of this nature will be borderline cases. As a suggestion try the Type I approach for calculating backwater first. Should the result appear unrealistic, repeat the backwater calculation using the Type II approach. It is more than likely that the difference in the two results will be great enough to readily spot the erratic one. Stating it another way, if the backwater for the Type II flow results in a lower value than for the Type I calculation, the flow definitely will be Type II.

6.3.6 Design Procedure

Table 6.5 gives a brief step-by-step outline of the procedure for determining a bridge waterway (that is, bridge length and deck level).

Table 6.5 - Design Procedure for Determining Bridge Waterway

Step	Design Procedure - *(Table 6.5)*
Step 1	Determine magnitude of flow at site for the design recurrence interval using Section 5, *Hydrology*, of this manual.
Step 2	Determine stage-discharge curve for the stream at the bridge site as follows : a. Plot representative cross-section of stream at Section 1 (see Figures 6.4 and 6.5). If channel is essentially straight and cross-section substantially uniform in the vicinity of the bridge, the natural cross-section at the bridge site may be used for this purpose. b. Subdivide cross-section according to marked changes in depth of flow and roughness. Assign values of Manning roughness coefficient ***n*** to each subsection (see Table 6.1 to 6.4). c. Calculate discharge in each subsection (method is demonstrated in the *Worked Example* in Section 6.3.7), for various stage heights. Sum discharge in subsections for each stage height and plot stage-discharge curve.
Step 3	Determine the stage height at the bridge site for the design discharge from the stage-discharge curve found above.
Step 4	Select velocity of flow through bridge opening to limit scour or encourage scour as required.
Step 5	Determine minimum length of bridge opening ***b*** required to pass design discharge assuming water surface is at stage height, as shown in Figure 6.4c. In cases involving spill through abutments, where the cross-section of the constrictions is irregular, convert the irregular cross-section to a regular trapezoid of equivalent area as shown on Figure 6.5c. Select a bridge deck level and trial length of bridge based upon minimum length of bridge opening and required length of spans.

Step	Design Procedure - *(Table 6.5)*
Step 6	Determine type of flow (see Section 6.3.1 b.) encountered as follows : a. Calculate average velocity of flow through bridge opening by dividing total discharge (design discharge) by cross-sectional area of flow (between abutments and below normal water surface level). b. Calculate the Froude number ***F*** in the constriction (see Section 6.2.1 d.). If ***F*** is less than ***1.0*** the flow is subcritical or Type I flow and backwater is estimated using the procedures in Section 6.3.2 a. . If ***F*** is greater than ***1.0*** the flow in the constriction is supercritical and the flow in the main channel of the natural cross-section should be checked by calculating the average velocity and Froude number in the main channel. If the Froude number in the main channel is also greater than ***1.0*** the flow is supercritical throughout or Type III flow and backwater should not occur (see Section 6.2.1 b.). If the Froude number is less ***1.0*** in the main channel, but greater than ***1.0*** in the constriction the flow is passing through critical and is either Type IIA or IIB. However, as indicated in Section 6.3.5 most Type II flow conditions are borderline cases and it is suggested that the backwater is calculated for the Type I and Type II cases and the lower value accepted. It should be noted also that scour will increase the bridge waterway and reduce velocities in the constriction, which will in many cases reduce the flow through the constriction from critical to subcritical.
Step 7	Calculate backwater using the procedure relevant to the type of flow encountered.

Step	Design Procedure - *(Table 6.5)*
Step 8	Having arrived at the stage height and backwater for the design discharge for the trial length of bridge, check the assumed deck level. a. At bridge opening check for clearance between water surface (assumed normal water level) and soffit of bridge deck. This clearance should not be less than 1 m. b. Along embankment where water level is the sum of the stage height and backwater, check that there is sufficient freeboard to the top of the embankment. This freeboard should not be less than 1 m. If there is insufficient clearance beneath the bridge lift the bridge deck level and if necessary recalculate the backwater. If there is insufficient free board to the top of the embankment, lift the embankment or reduce the backwater by using a larger bridge.

6.3.7 Worked Example

The worked example in Table 6.8 gives a step-by-step design procedure for the crossing detailed in Table 6.6.

a. Details of Crossing

Table 6.6 - Worked Example - Details of Crossing

Detail	Description See Figure 6.19, 6.20 & 6.21
Detail 1	Stream is essentially straight, the cross-section relatively constant in the vicinity of the bridge, and the crossing is normal to the general direction of flow.
Detail 2	Average slope of stream in vicinity of bridge, s = 0.00042 m/m.
Detail 3	Bridge substructure to be constructed utilising 5 number 500 mm diameter piles at each pier.
Detail 4	Bridge abutments to be spill through type with 1.5:1 slope.

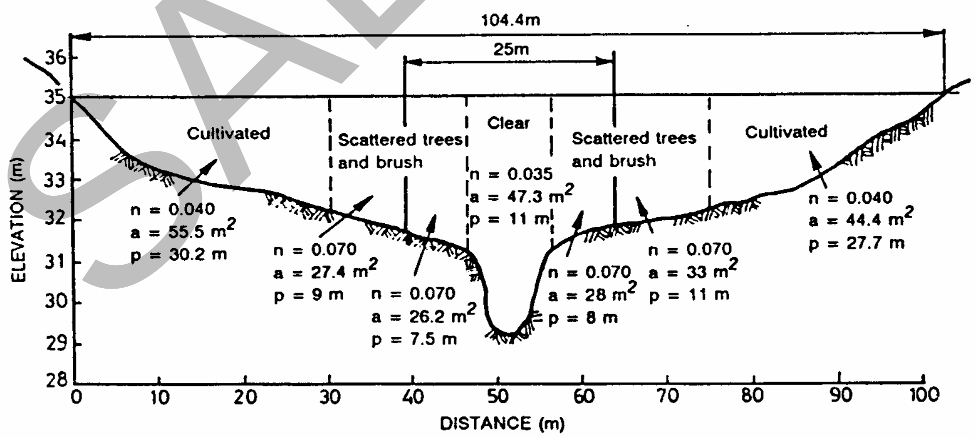

Figure 6.19 - Cross-Section of Stream at Bridge Site (looking upstream)

b. Design Procedure

Table 6.7 - Worked Example - Design Procedure

Step	Design Procedure - *(Table 6.7)*
Step 1	Design discharge is 220 m^3/s.
Step 2	Determine stage-discharge curve. a. Figure 6.19 shows cross-section of river at bridge site. b. Subdivision of cross-section and values of Manning's ***n*** are also shown on Figure 6.19. c. For simplicity only a stage height of 35.0 m will be investigated. Table 6.8 shows the calculation of the discharge for a stage height of 35.0 m. The discharge for other stage heights are treated similarly and the stage discharge curve drawn as shown in Figure 6.20.
Step 3	From Figure 6.20 it can be seen that the stage-height at the bridge site for the design discharge of 220 m^3/s is 35.0 m.
Step 4	Assuming a maximum average velocity through the bridge opening, $V_{max.} = 2.2\ m/s$ Without scour, minimum length of bridge opening ***b*** required to pass design discharge. $b = \frac{Q}{V_{max.}} \times \frac{1}{\bar{y}}$ where $\bar{y}$ = average depth of flow in the constriction = 4.2 m $b = \frac{220}{2.2} \times \frac{1}{4.2} = 23.8\ m$ Say a length of bridge opening of 25 m positioned as shown on Figure 6.19 . Assuming a deck level of 36.5 m and positioning the spill through abutments to maintain the same waterway area, try a bridge length of 34 m (10 m - 14 m - 10 m span configuration) with a structural depth of bridge deck of 1 m as shown in Figure 6.21 .

Step	Design Procedure - *(Table 6.7)*
Step 5	Determine type of flow. Calculate Froude number F in the constriction : $$F = \frac{V}{\sqrt{gd}}$$ V = average velocity through bridge opening = 2.2 m/s d = y = 4.2 m Flow is subcritical or Type I flow.
Step 6	Calculate conveyance in each subsection for the design discharge as shown in Table 6.8.
Step 7	Determine kinetic energy coefficient α_1. a. Calculate velocity, v and qv^2 in each subsection as shown in Table 6.8. b. Calculate average velocity V_{n1} in channel section. $$V_{n1} = \frac{Q}{A_{n1}} = \frac{220}{261.4} = 0.84 m/sec$$ c. Then, $$\alpha_1 = \frac{\sum(qv^2)}{QV_{n1}^2} = \frac{246.2}{220 \times (0.84)^2} = 1.59$$
Step 8	Calculate bridge opening ratio, M (see Table 6.8).

<table>
<tr><th>Step</th><th>Design Procedure - (Table 6.7)</th></tr>
<tr><td>Step 9</td><td>
Determine total backwater coefficient K^*.

a. Determine base curve coefficient K_b from Figure 6.8 . With $M = 0.5$, $K_b = 1.12$

b. Determine incremental backwater coefficient K_p for effect of piers.

From Figure 6.21 gross water area of the constriction $A_{n2} = 98.9$.

For 2 number piers comprising 5 number 500 mm diameter piles, area of obstruction

$$A_p = (0.5 \times 3.5) \times 2 = 3.5\ m^2$$
$$J = \frac{A_p}{A_{n2}} = \frac{3.5}{98.9} = 0.035$$
From Figure 6.9a, for $M = 1.0$, $\Delta K = 0.12$ and Figure 6.9b, for $M = 0.5$, $\sigma = 0.68$

$$\Delta K_p = \Delta K\ \sigma = 0.12 \times 0.68 = 0.082$$
c. As eccentricity is less than 20%, $\Delta K_c = 0$.

d. For no skew, $\Delta K_s = 0$.

e. Hence, total backwater coefficient

$$\begin{aligned} K^* &= K_b + \Delta K_p + \Delta K_c + \Delta K_s \\ &= 1.12 + 0.082 + 0.0 + 0.0 \\ &= 1.202 \end{aligned}$$
</td></tr>
</table>

Step	Design Procedure - *(Table 6.7)*
Step 9 *(continued)*	f. Calculate backwater h^*_1. Average velocity in constriction. $$V_{n2} = \frac{Q}{A_{n2}} = \frac{220}{98.9} = 2.22\ m/\text{sec}$$ and $$\frac{V_{n2}^2}{2g} = \frac{2.22^2}{2 \times 9.81} = 0.25$$ From Figure 6.7 for $\alpha_1 = 1.59$ and $M = 0.5$ $$\alpha_2 = 1.3$$ Using Equation (6.15) approximate backwater will be $$K^* \ \alpha_2 \ \frac{V_{n2}^2}{2g} = 1.202 \times 1.3 \times 0.25 = 0.391\ m$$ Substituting values in the second half of Equation (6.14) with $$A_1 = A_{n1} + h_1^* \times W$$ where W = width of flow (m) $$A_1 = 261.4 + (0.391 \times 104.4) = 302.22\ m^2$$ Then, $$\alpha_1 \left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right] \frac{V_{n2}^2}{2g}$$ $$= 1.59 \left[\left(\frac{98.9}{261.4}\right)^2 - \left(\frac{98.9}{302.2}\right)^2\right] 0.25 = 0.056\ m$$ Then the total backwater produced by the bridge $$= 0.391 + 0.056 = 0.447\ m$$

Step	Design Procedure - *(Table 6.7)*
Step 10	Check assumed deck level of 36.5 m. a. At bridge opening (see Figure 6.21) Stage height 35.0 Clearance under bridge deck 0.5 Structural depth of bridge deck 1.0 Minimum deck level <u>36.50</u> m b. Along embankment where water level is affected by backwater Stage height 35.0 Backwater 0.45 Minimum freeboard 0.5 Minimum deck level <u>35.95</u> m The first of these calculations (Step 10 a.) governs and bridge deck level of 36.5 m is acceptable.

Table 6.8 - Worked Example - Properties of Natural Stream
(For Stage Height 35.0 m)

Subsection		Step 2 - Table 6.8						Step 6	Step 7	
		n	a m^2	p m	r = a/p m	$r^{2/3}$	q m^3/s	K = q/ $s^{1/2}$	v = q/a m/s	qv^2
Q_A	0 - 30	0.040	55.3	30.2	1.83	1.50	42.4	2069.2	0.77	25.1
	30 - 39	0.070	27.4	9.0	3.04	2.10	16.8	822.2	0.61	6.2
Q_B	39 - 46.5	0.070	26.2	7.5	3.49	2.30	17.7	861.7	0.67	7.9
	46.5 - 56	0.035	47.3	11.0	4.30	2.64	73.2	3573.6	1.55	175.9
	56 - 64	0.070	28.0	8.0	3.50	2.31	18.9	922.1	0.67	8.5
Q_C	64 - 75	0.070	33.0	11.0	3.00	2.08	20.1	980.6	0.61	7.5
	75 - 102.5	0.040	44.2	27.7	1.60	1.37	30.9	1508.9	0.70	15.1
		A_{n1} = 261.40				Q = 220.00			Σqv^2 = 246.20	

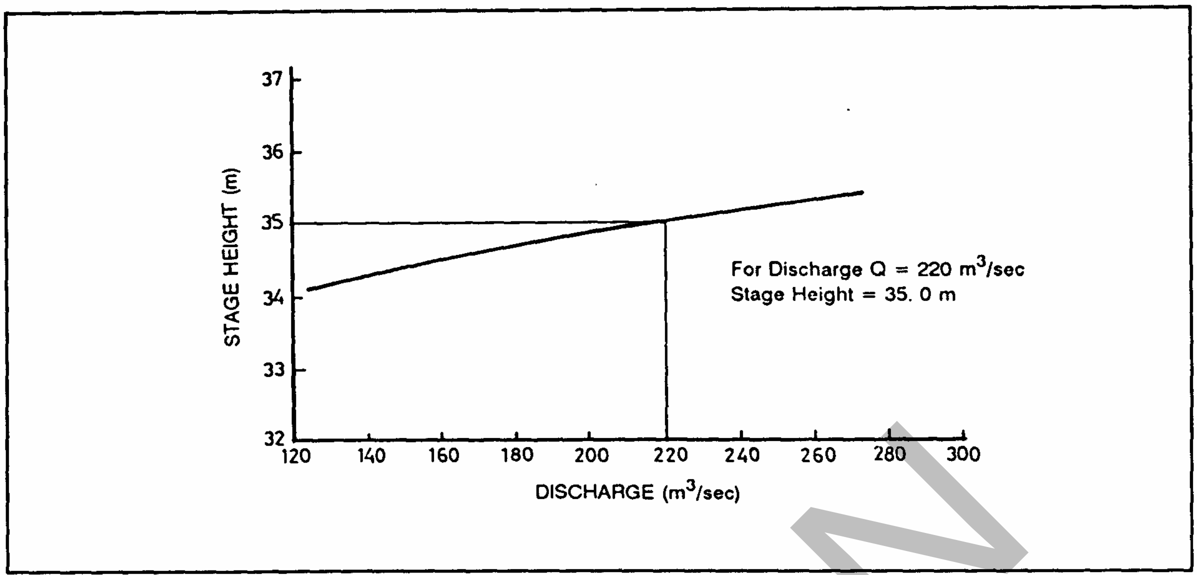

Figure 6.20 - Worked Example - Stage-Discharge Curve

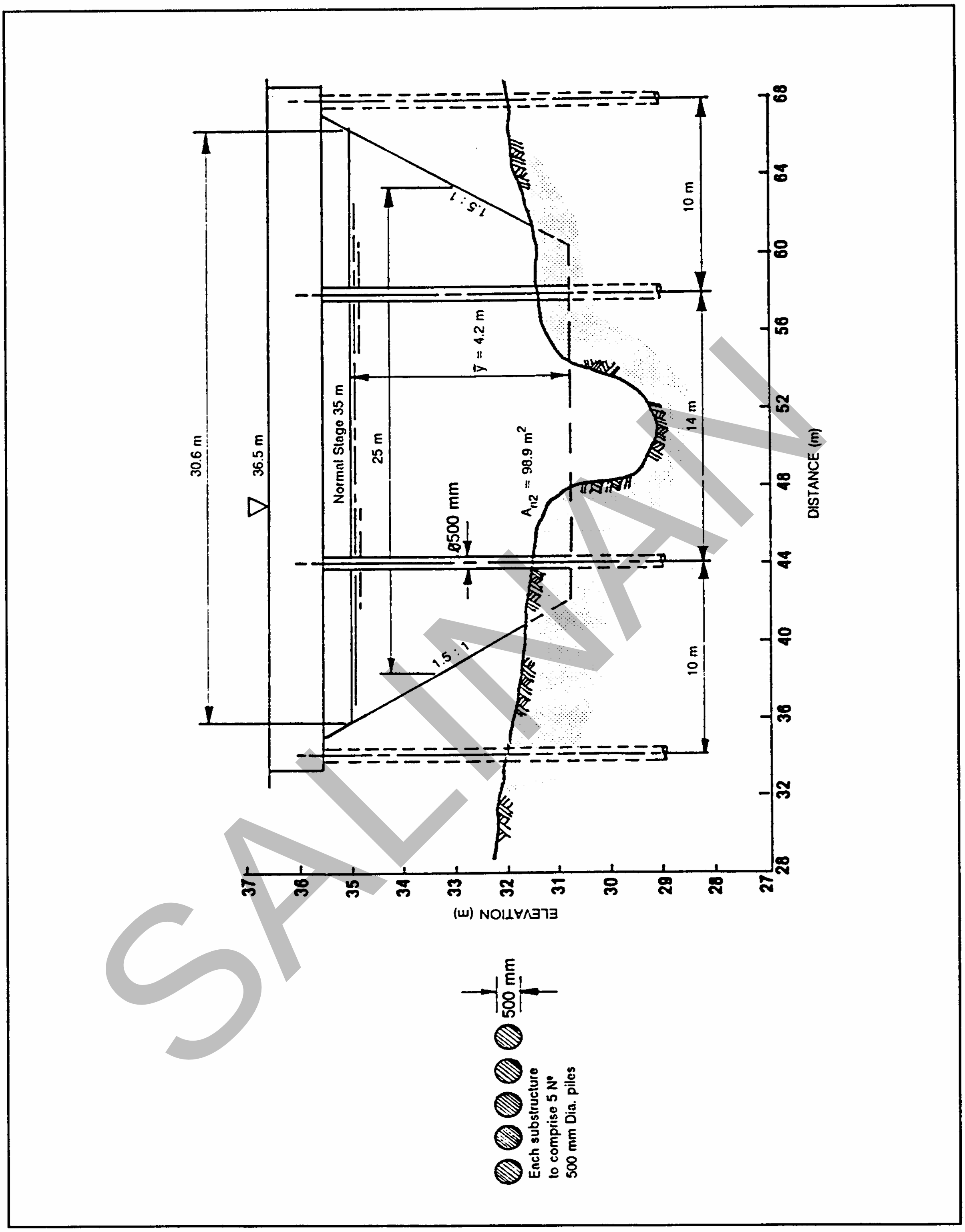

Figure 6.21 - Worked Example - Cross-Section at Bridge

6.4 CULVERT WATERWAY DESIGN

6.4.1 Scope

This section of the manual contains a brief discussion of the hydraulics of conventional culverts and nomographs for selecting a culvert size for a given set of conditions.

6.4.2 Types of Flow

Laboratory tests and field observations show two major types of culvert flow :

- flow with inlet control, and
- flow with outlet control.

For each type of control, different factors and formulae are used to calculate the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness and length of the culvert barrel.

It is possible by involved hydraulic calculations to determine the probable type of flow under which a culvert will operate for a given set of conditions. The need for making these calculations may be avoided however, by calculating headwater depths from the nomographs included in this paper for both inlet control and outlet control, and then using the higher value to indicate the type of control and the headwater dept. This method of determining the type of control is accurate except for a few cases where the headwater is approximately the same for both types of control.

Both inlet control and outlet control types of flow are discussed briefly in the following sections and procedures for the use of the nomographs are given.

6.4.3 Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater, ***HW***, and the entrance geometry, including the barrel shape and cross-sectional area, and the type of inlet edge. Sketches of inlet control flow for both unsubmerged and submerged projecting entrances are shown in Figures 6.22a and 6.22b. Figure 6.22c shows a mitered entranced flowing under a submerged condition with inlet control.

In inlet control the roughness and length of the culvert barrel and outlet conditions (including depth of tailwater) are not factors in determining culvert capacity. An increase in barrel slope reduces headwater to a small degree and any correction for slope can be neglected for conventional or commonly used culverts flowing with inlet control.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth (or headwater ***HW***) is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth + velocity head). Because of the low velocities in most entrance pools and the difficulty in determining the velocity head for all flows, the water surface and the energy line at the entrance are assumed to be coincident, thus the headwater depths given by the inlet control charts in this manual can be higher than will occur in some installations. For the purposes of measuring headwater, the culvert invert at the entrance is the low point in the culvert opening at the beginning of the full cross-section of the culvert barrel.

6.4.4 Outlet Control

Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it, (see Figure 6.23). If the entire cross-section of the barrel is filled with water for the total length of the barrel, the culvert is said to be in full flow or flowing full, Figures 6.23a and 6.23b. Two other common types of outlet-control flow are shown in Figures 6.23c and 6.23d. The procedures given in this paper provide methods for the accurate determination of headwater depth for the flow conditions shown in Figures 6.23a, 6.23b and 6.23c. The method given for the part full flow condition, Figure 6.23d, gives a solution for headwater depth that decreases *n* accuracy as the headwater decreases.

The head H (Figure 6.23a) or energy required to pass a given quantity of water through a culvert flowing in outlet control with the barrel flowing full throughout its length is made up of three major parts. These three parts are usually expressed in metres of water and include a velocity head H_v, an entrance loss H_e, and a friction loss H_f. This energy is obtained from ponding of water at the entrance and expressed in equation form

$$H = H_v + H_e + H_f \tag{6.30}$$

The velocity head H_v equals $V^2/2g$, where V is the mean or average velocity in the culvert barrel. (The mean velocity is the discharge Q, in m³/sec, divided by the cross-sectional area A, in m², of the barrel.)

The entrance loss H_e depends upon the geometry of the inlet edge. This loss is expressed as a coefficient k_e times the barrel velocity head or $H_e = k_e\ V^2/2g$. The entrance loss coefficient k_e for various types of entrances when the flow is outlet control are given in Table 6.9.

The friction loss H_f is the energy required to overcome the roughness of the culvert barrel. H_f can be expressed in several ways. Since most bridge engineers are familiar with Manning's n the following expression is used :

$$H_f = \left[\frac{19.6\ n^2\ L}{R^{1.33}}\right]\frac{V^2}{2g} \tag{6.31}$$

where
- n = Manning's friction factor (see nomographs for values)
- L = length for culvert barrel (m)
- V = mean velocity of flow in culvert barrel (m/sec)
- g = acceleration of gravity, ***9.81*** (m/sec²)
- R = hydraulic radius or A/WP (m)

where
- A = area of flow for full cross-section (m²)
- WP = wetted perimeter (m)

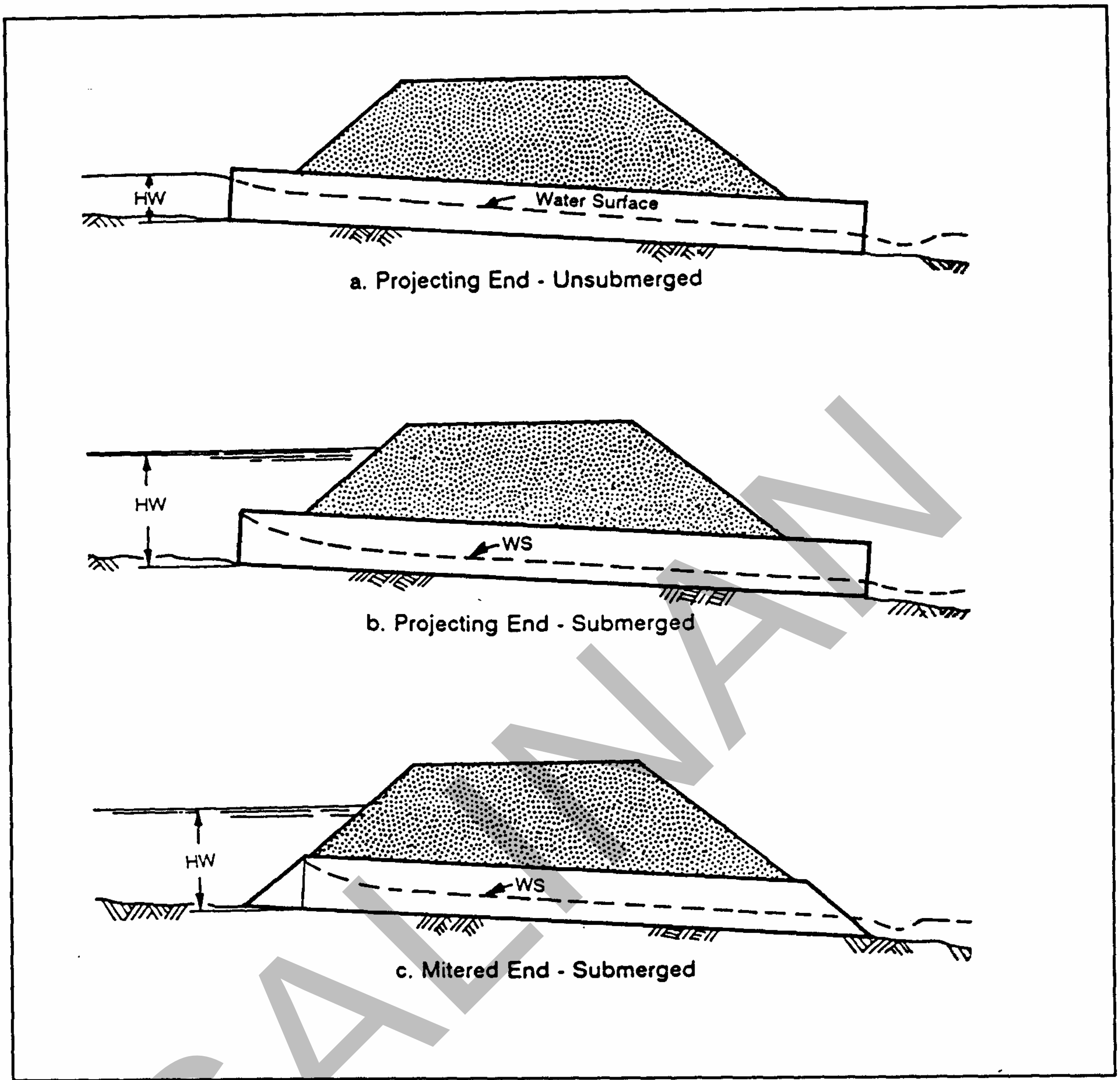

Figure 6.22 - Culvert with Inlet Control

Substituting in Equation 6.30 and simplifying, we get for full flow

$$H = \left[1 + k_e + \frac{19.6n^2L}{R^{1.33}}\right]\frac{V^2}{2g} \qquad (6.32)$$

Figure 6.24 shows the terms of Equation 6.32, the energy line, the hydraulic grade line and the headwater depth, ***HW***. The energy line represents the total energy at any point along the culvert barrel. The hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length. The energy line and the pressure line are parallel over the length of the barrel except in the immediate vicinity of the inlet where the flow contracts and re-expands. The difference in elevation between these two lines is the velocity head, $V^2/2g$.

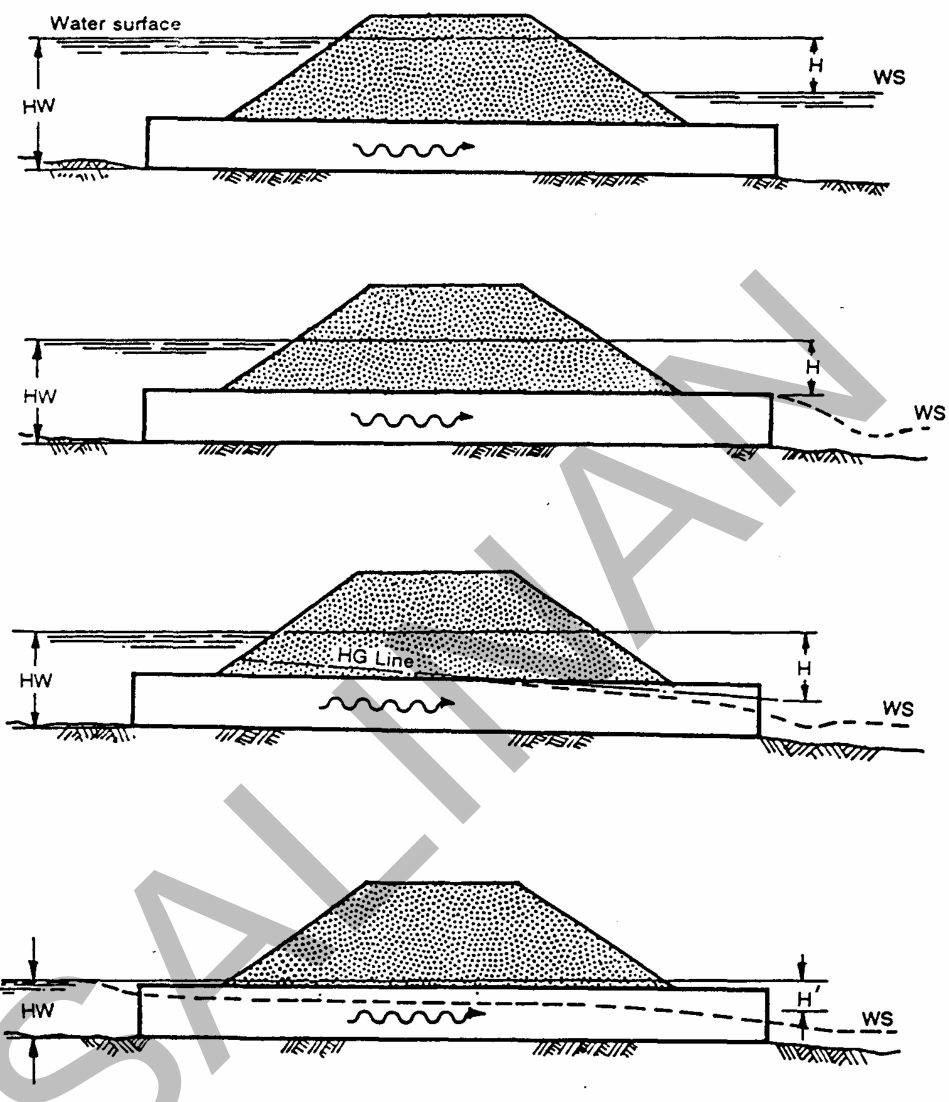

Figure 6.23 - Culvert with Outlet Control

Equation 6.32 can be solved readily by the use of the full-flow nomographs, Figures 6.30 to 6.33. The equations shown on these nomographs are the same as Equation 6.32 expressed in a different form. Each nomograph is drawn for a single value of ***n*** as noted on the respective chart. These nomographs can be used for other values of ***n*** by modifying the culvert length.

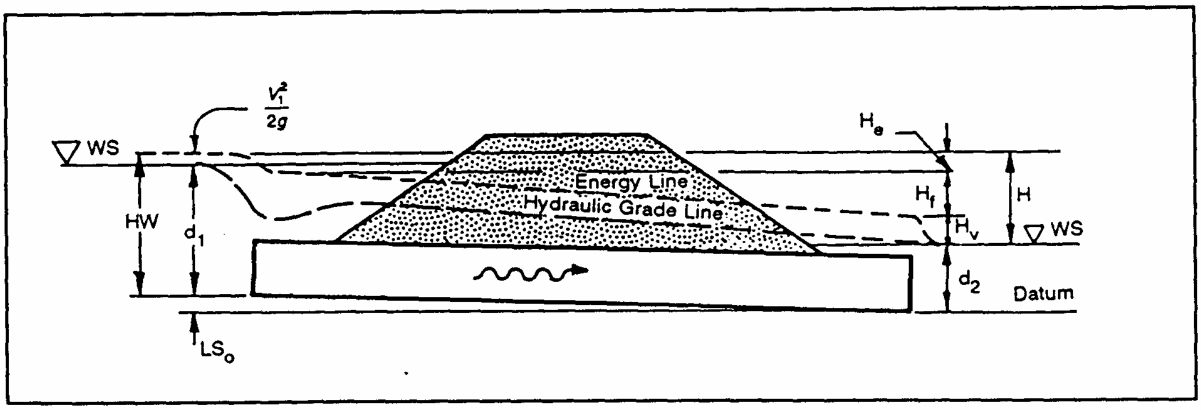

Figure 6.24 - Terminology for Full Flow Conditions

Finding the value of H from a nomograph is not the complete solution for outlet control type of flow. headwater must be determined and other factors such as slope of the culvert barrel and outlet conditions enter into this calculation.

The value of H in metres must be measured from some *control* elevation at the outlet. This *control* elevation is dependent on the rate of discharge or the elevation of the water surface of the tailwater. For simplicity a value h_o is used as the distance in metres from the culvert invert (flow line) at the outlet to the *control* elevation. The following equation is used to calculate headwater (HW) :

$$HW = h_o + H - LS_o \qquad (6.33)$$

where S_o is the slope of the flow line in m per m and all terms are in metres. The determination of h_o is discussed in the following paragraphs for the various flow conditions at the outlet.

If the water surface in the outlet channel (tailwater elevation) is at or above the top of the barrel at the outlet (Figure 6.23a) the solution for HW is simple. The TW depth is equal to h_o and the relationship of HW to the other terms in Equation 6.33 are illustrated in Figure 6.25.

If the tailwater elevation is below the top or crown of the culvert at the outlet, the determination of h_o for a given discharge and size of culvert is more difficult. Figure 6.23 (b, c and d), h_o is found by comparing two values :

- TW depth in the outlet channel and

- $d_c + D/2$ and setting h_o equal to the larger of these values.

The fraction $d_c + D/2$ is a simplified means of calculating h_o when the tailwater is low and the discharge does not fill the culvert barrel at the outlet. In this fraction d_c is critical depth as determined from Figures 6.34 and 6.35 and D is the culvert height. The value of d_c should never exceed D, making the upper limit of this fraction equal to D. The sketch in Figure 6.26 shows the terms of Equation 6.33 for the cases discussed above.

From more rigorous solutions it has been found that Equation 6.33 gives accurate answers

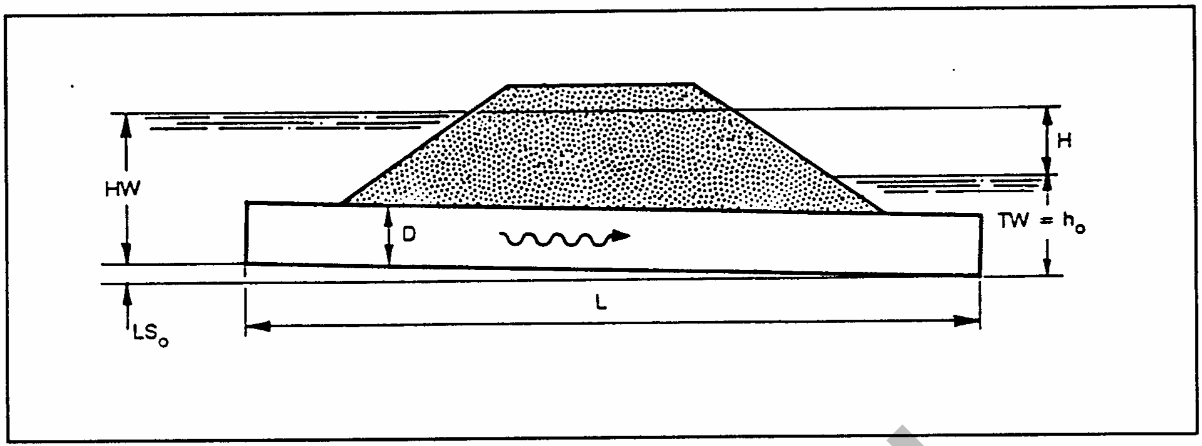

Figure 6.25 - Tailwater At or Above Top of Culvert

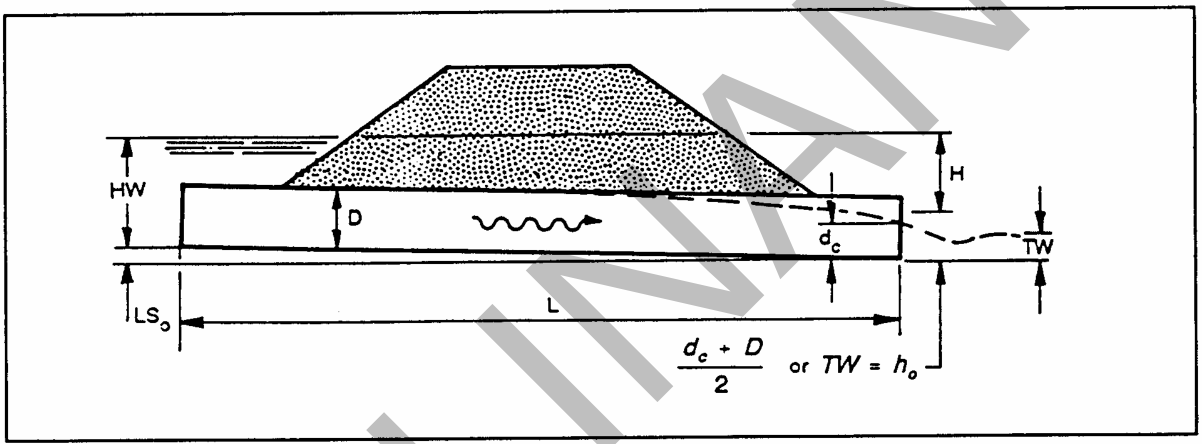

Figure 6.26 - Tailwater Below Top of Culvert

if the culvert flows full for a part of the barrel length as illustrated by Figure 6.26. This condition of flow will exist if the headwater as determined by Equation 6.33 is equal to or greater than the quantity :

$$D + (1 + k_e) \frac{V^2}{2g} \qquad (6.34)$$

where V is the mean velocity for the full cross-section of the barrel; k_e, the entrance loss coefficient; and D, the culvert height. If the headwater drops below this point the water surface will be free throughout the culvert barrel as in Figure 6.23d and Equation 6.33 gives answers with some error as explained in the next paragraph.

In case Figure 6.23d, Equation 6.33 is used to solve for *HW* when a free water surface exists through the barrel. Such a calculation does not give a true value since the only correct way of finding *HW* in this case is by a backwater calculation starting at the culvert outlet. However, Equation 6.33 will give answers of sufficient accuracy for design purposes if the headwater is limited to values greater than *0.75D*. *H'* is used in Figure 6.23d to show that the head loss here is an approximation of *H*. Culvert capacity charts found in *Hydraulic Engineering Circular No. 10* (Reference 6.9) give a more accurate and easy solution for this free surface flow condition.

Although the procedure given in this section is primarily for use in selecting a size of culvert to pas a given discharge at a given headwater, a better understanding of culvert operation is gained by plotting performance curves through some range of discharges and barrel slopes. Such curves can also be used to compare different sizes and types of culverts.

6.4.5 Tailwater Depth

The depth of tailwater is important in determining the hydraulic capacity of culverts flowing with outlet control. In many cases the downstream channel is of considerable width and the depth of water in the natural channel is less than the height of water in the outlet end of the culvert barrel, making the tailwater ineffective as a control, so that its depth need not be calculated to determine culvert discharge capacity of headwater. There are instances however, where the downstream water-surface elevation is controlled by a downstream obstruction or backwater from another stream. A field inspection of all major culvert locations should be made to evaluate downstream controls and determine water stages.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation (see Section 6.2) if the channel is reasonably uniform in cross-section, slope and roughness.

Values of ***n*** for natural streams in Manning's formula may be found in Section 6.2. If the water surface in the outlet channel is established downstream controls other means must be found to determine the tailwater elevation. Sometimes this necessitates a study of the stage-discharge relationship of another stream into which the stream in question flows or the securing of data on reservoir elevations if a storage dam is involved.

6.4.6 Velocity of Flow

A culvert, because of its hydraulic characteristics, increases the velocity of flow over that in the natural channel. High velocities are most critical just downstream from the culvert outlet and the erosion potential from the energy in the water is a feature to be considered in culvert design.

Energy dissipators for channel flow have been investigated in the laboratory and many have been constructed, especially in irrigation channels. Some of these structures have been modified and at least several hundred have been constructed at the outlets of culverts. All energy dissipators add to the cost of a culvert and engineers should consider using them only when required to prevent a large scour hole or as remedial construction.

The judgement of engineers working in a particular area is required to determine the need for energy dissipators at culvert outlets. As an aid in evaluating this need it is suggested that the outlet velocities be calculated. These calculated velocities can be compared with outlet velocities or other sizes and types of culverts and with the natural channel velocities. A change in size of culvert does not change outlet velocities appreciably in most cases. Average outlet velocities for culverts flowing with inlet control may be approximated by calculating the normal velocity for the culvert cross-section using a uniform flow equation.

Since the depth of flow is not known the use of tables or charts is recommended in solving this equation. The outlet velocity for inlet control calculated in this manner will be high for culverts having a length-depth ratio less than say ***20***. The shorter culverts velocities will be between those calculated by a uniform flow equation and those occurring at critical depth.

In outlet control, the average outlet velocity will be the discharge divided by the cross-sectional area of flow at the outlet. This flow area will be between that corresponding to critical depth and the full area of the pipe, depending upon the tailwater conditions.

6.4.7 Design Procedure

Table 6.9 - Design Procedure for Determining Culvert Waterway

Step	Design Procedure - *(Table 6.9)*
Step 1	*List design data* a. Design discharge in m^3/sec with associated design frequency, that is, Q_{50}, Q_{20} etc. b. Approximate length of culvert ***L***, in metres. c. Slope of culvert in m per m. d. Allowable headwater. e. Mean and maximum flood velocities in natural stream. f. Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape, and entrance type.
Step 2	*Determine the first trial size culvert.* Since the procedure is one of trial and error, the initial trial size can be determined in several ways : a. By arbitrary selection. b. By using an approximating equation such as $A = Q/V$ from which the trial culvert dimensions are determined. c. By using inlet control nomographs (Figures 6.27 to 6.29) for the culvert type selected. If this method is used and ***HW/D*** must be assumed, say $HW/D = 1.5$, and using the given ***Q*** a trial size is determined. If any trial size is too large because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge equally between the number of barrels used.

Step	Design Procedure - *(Table 6.9)*
Step 3	*Find headwater depth for trial size culvert.* a. Assuming *inlet control* : i. Using the trial size from Step 2, find the headwater depth HW by use of the appropriate inlet control nomograph. Tailwater conditions are to be neglected in this determination. ***HW*** in this case is found by multiplying ***HW/D*** obtained from the nomographs by the height of culvert ***D***. b. Assuming *outlet control* : i. Approximate the depth of tailwater ***TW***, in m above the invert at the outlet for the design flood condition in the outlet channel. ii. For tailwater elevation equal to or greater than the top of the culvert at the outlet (Figure 6.25) set h_o equal to ***HW*** and find ***HW*** by the following equation : $HW = H + h_o - LS_o$ ***H*** is the head loss in metres determined from the appropriate nomographs (Figures 6.30 to 6.33) representing Equation 6.32). $H = \left[1 + k_e + \frac{19.6\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$ where k_e = entrance loss coefficient n = Manning's friction coefficient R = hydraulic radius V = mean velocity in the barrel iii. For the tailwater elevation less than the top of the culvert at the outlet (Figure 6.26), find headwater ***HW*** by the above equation as in b. ii. above except that $h_o = d_c + D/2$ or ***TW***, whichever is the greater. d_c is the critical depth of flow obtained from Figures 6.34 and 6.35.

Step	Design Procedure - *(Table 6.9)*
Step 3 (continued)	c. Compare the headwaters found in Step 3 a. and b. The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected. d. If outlet control governs and the headwater is higher than acceptable, select a larger trial size and find ***HW*** as in Step 3 b. Inlet control need not be checked, since the smaller size was satisfactory for this control as determined in Step 3 a.
Step 4	*Try a culvert of another type or shape and determine size and **HW** by the above procedure.*
Step 5	*Calculate outlet velocities for size and types to be considered in selection and determine need for channel protection.* a. If *outlet control governs* in Step 3 c., outlet velocity equals Q/A_o where A_o is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or ***TW*** is less than the height of the culvert barrel, use A_o corresponding to d_c or tailwater depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel. b. If *inlet control governs* in Step 3 c., outlet velocity can be assumed to equal normal velocity in open-channel flow in the barrel as calculated by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.
Step 6	*Record final selection of culvert.* Include the following : • culvert size • culvert type • required headwater • outlet velocity • economic justification.

Table 6.10 - Entrance Loss Coefficients for Culverts

1. Concrete Pipe Culvert			Coeff k_e
a.	Projecting from fill, socket end (groove end)		0.2
b.	Projecting from fill, square cut end		0.5
c.	Headwall or headwall and wing walls		
	i.	Socket end of pipe (groove end)	0.2
	ii.	Square-edge	0.5
	iii.	Rounded (radius = D/12)	0.2
d.	Mitered to conform to fill slope		0.2
e.	End section conforming to fill slope (see Note 3)		0.5
2. Corrugated Metal Type Culvert			**Coeff k_e**
a.	Projecting from fill (no headwall)		0.9
b.	Headwall or headwall and wingwalls		
	i.	Square-edge	0.5
c.	Mitered to conform to fill slope		0.7
d.	End-section conforming to fill slope (see Note 3)		0.5
3. Reinforced Concrete Box Culvert			**Coeff k_e**
a.	Headwall parallel to embankment (no wingwalls)		
	i.	Square-edged on 3 edges	0.5
	ii.	Rounded on 3 edges to radius of $^1/_{12}$ barrel dimension	0.2
b.	Wingwalls at 30° to 75° to barrel		
	i.	Square-edged at crown	0.4
	ii.	Crown edge rounded to radius of $^1/_{12}$ barrel dimension	0.2
c.	Wingwalls at 10° to 25° to barrel		
	i.	Square-edge at crown	0.5
d.	Wingwalls parallel (extension of sides)		
	i.	Square-edged at crown	0.7

NOTES

1. *Coefficient k_e to apply to velocity head $V^2/(2g)$ for determination of head loss at entrance to a culvert operating full or partly full with control at the outlet.*

2. *Entrance head loss $H_e = k_e\ V^2/(2g)$.*

3. ***End section conforming to fill slope** refers to the sections available from manufacturers.*

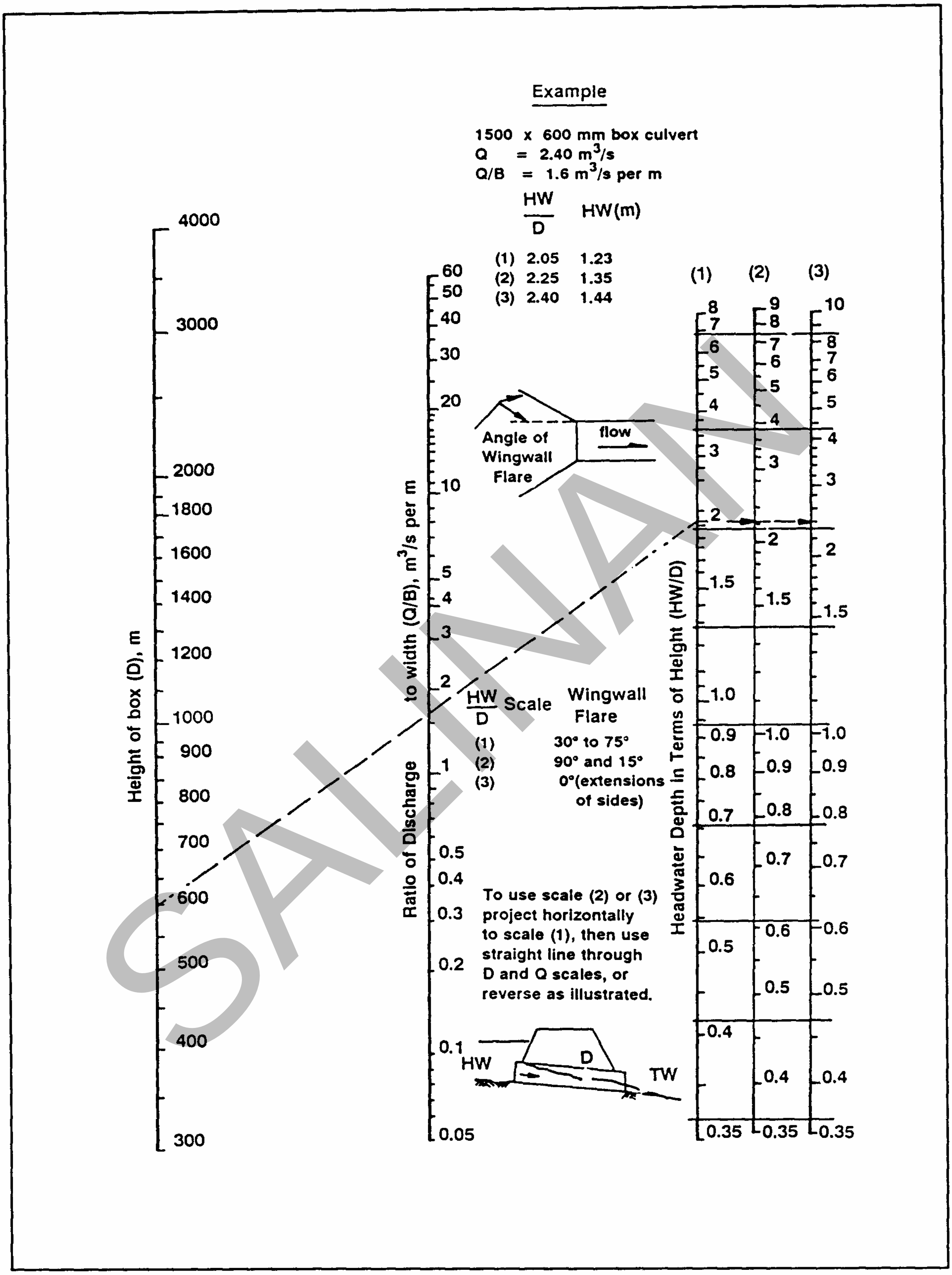

Figure 6.27 - Headwater Depth for Box Culverts with Inlet Control

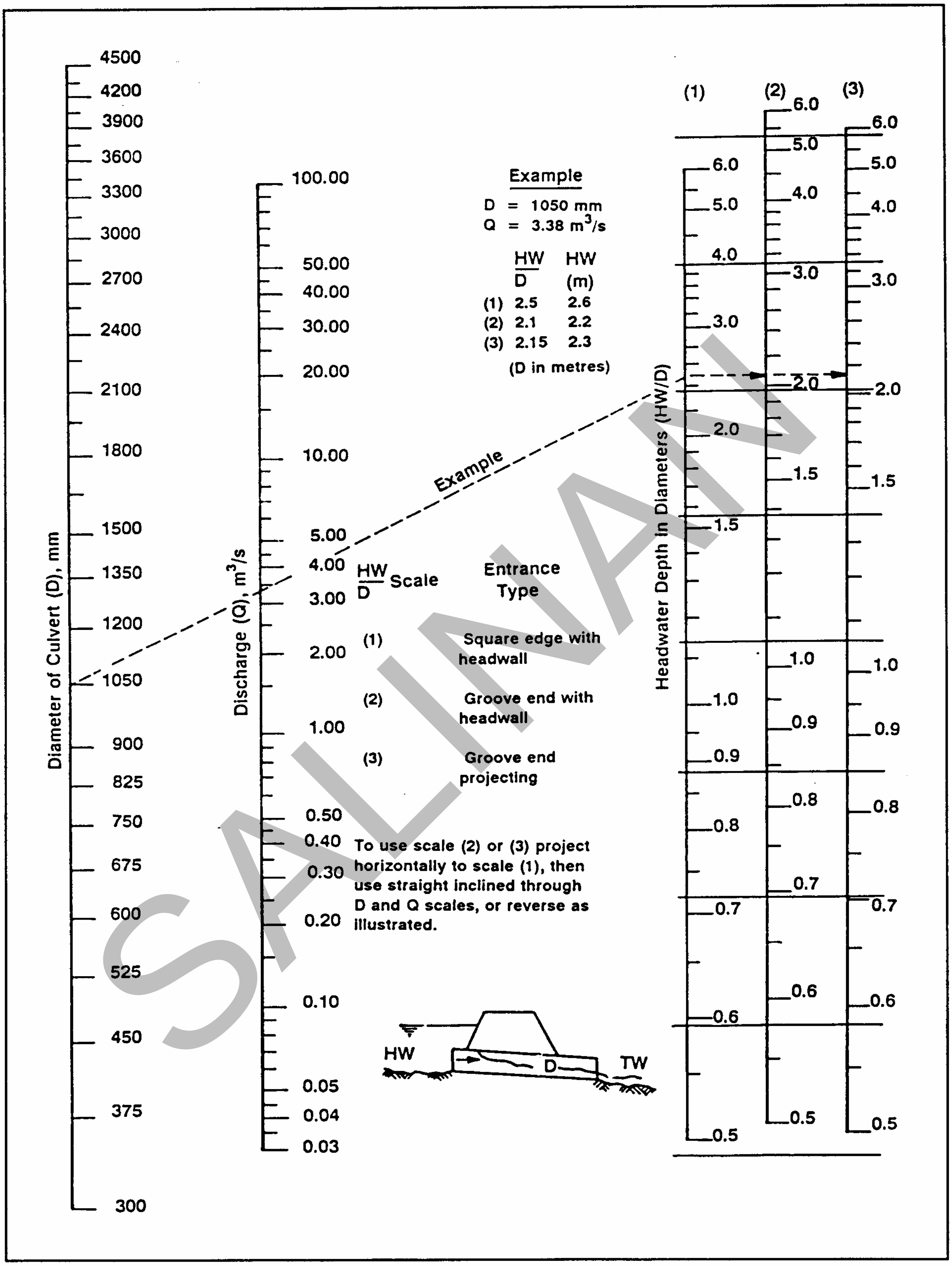

Figure 6.28 - Headwater Depth for Concrete Pipe Culverts with Inlet Control

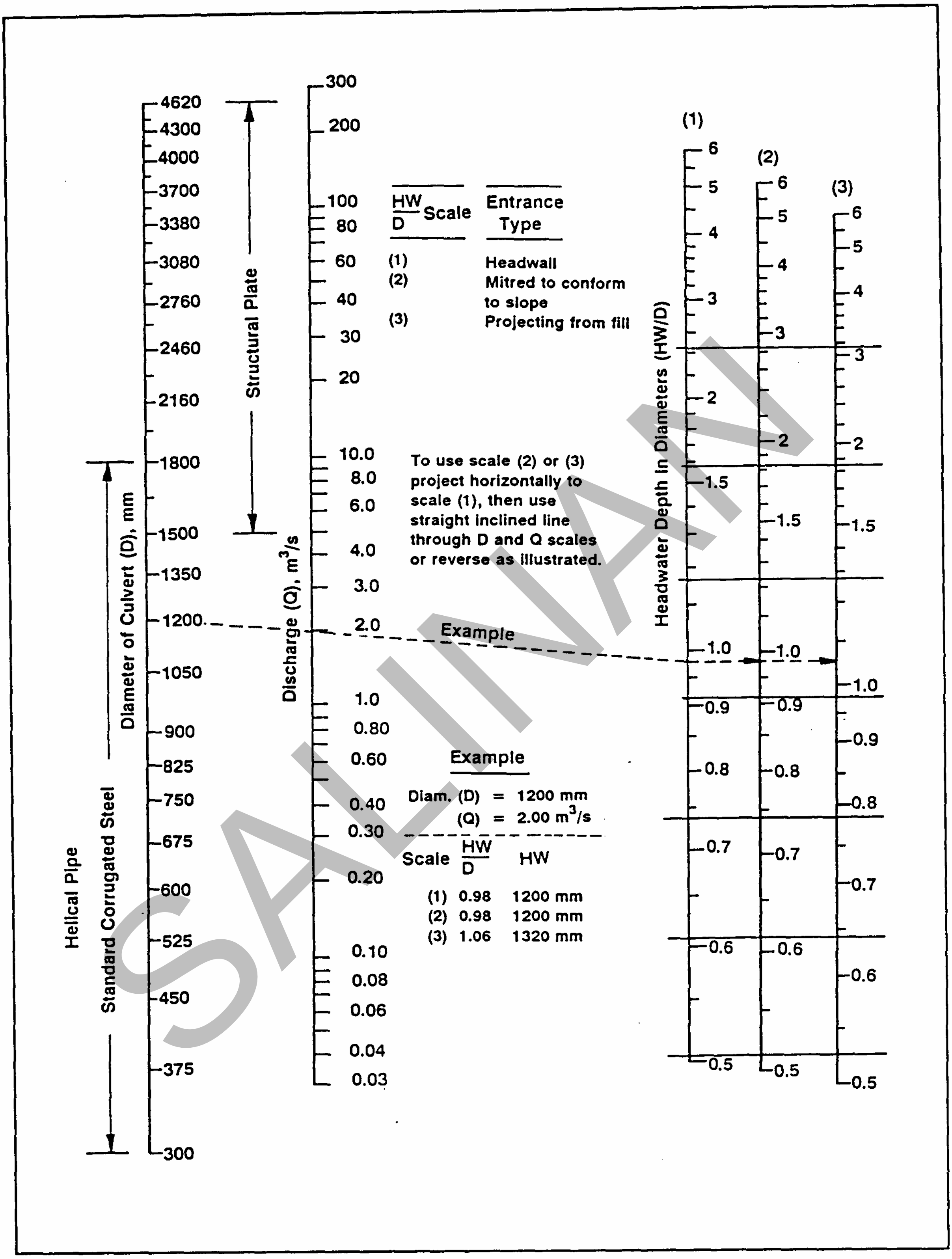

Figure 6.29 - Headwater Depth for Corrugated Steel Pipe Culverts with Inlet Control

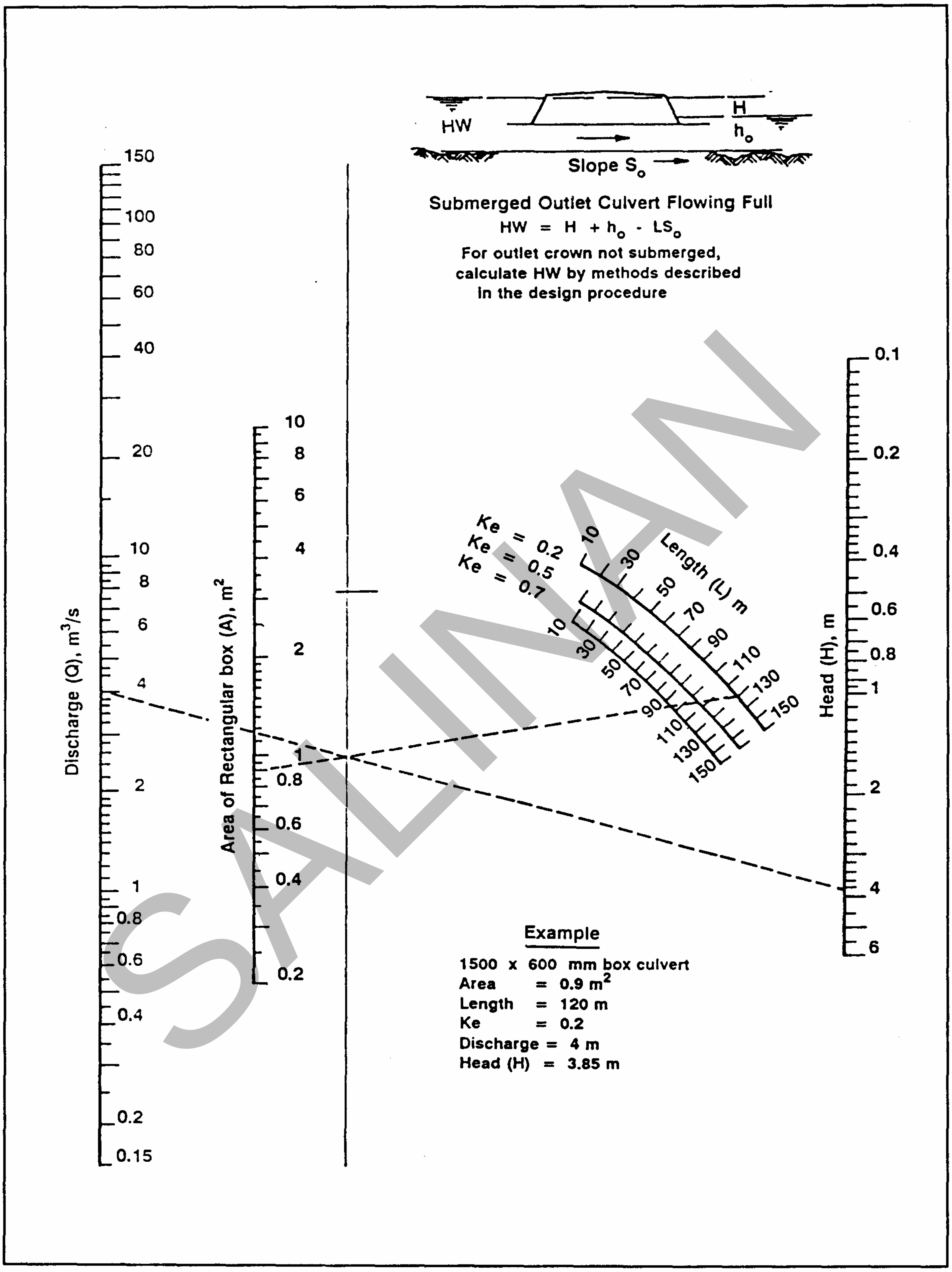

Figure 6.30 - Headwater Depth
for Concrete Box Culverts Flowing Full with Outlet Control $n = 0.012$

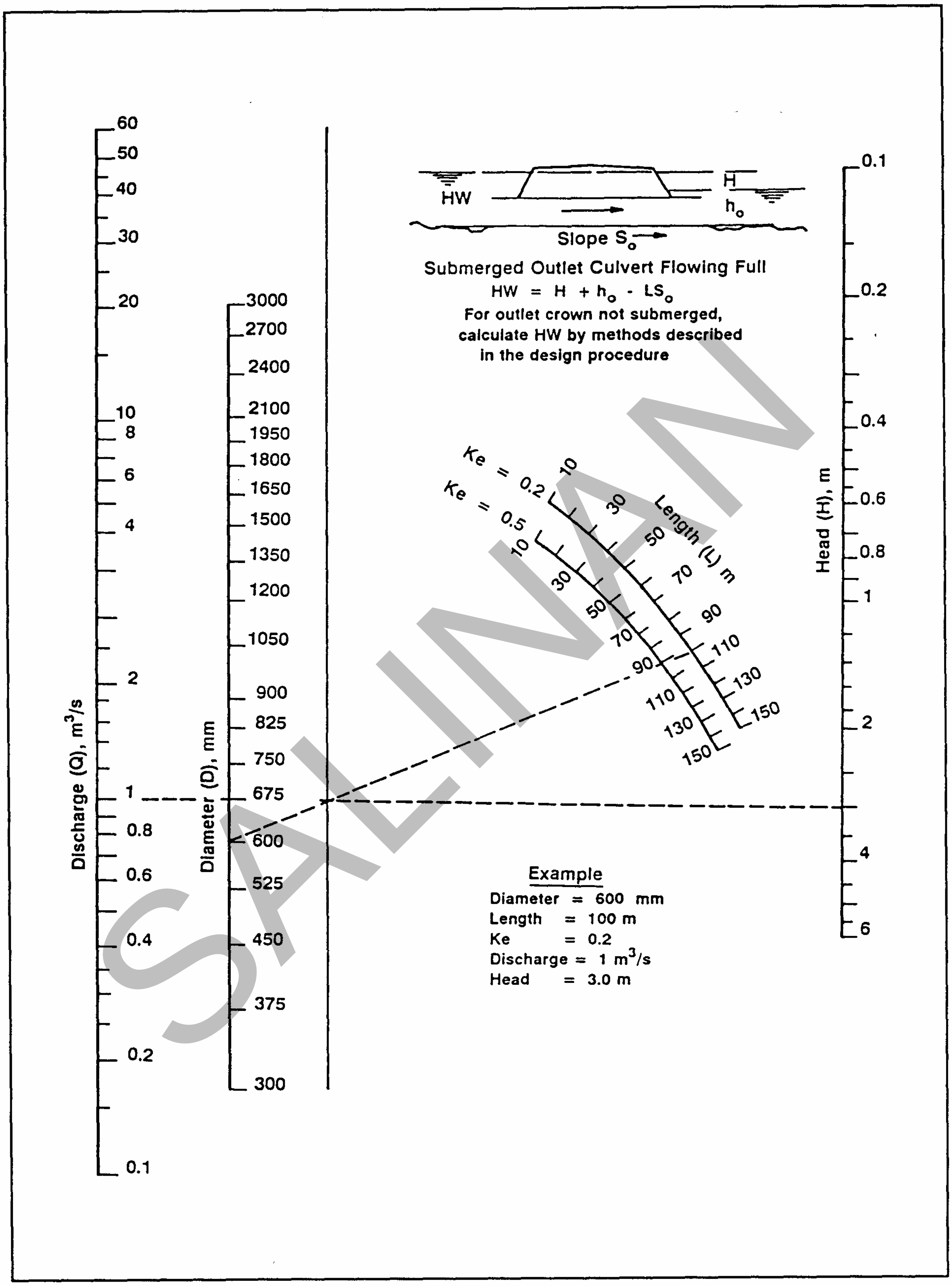

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Figure 6.31 - Headwater Depth for Concrete Pipe Culverts Flowing Full with Outlet Control n = 0.012

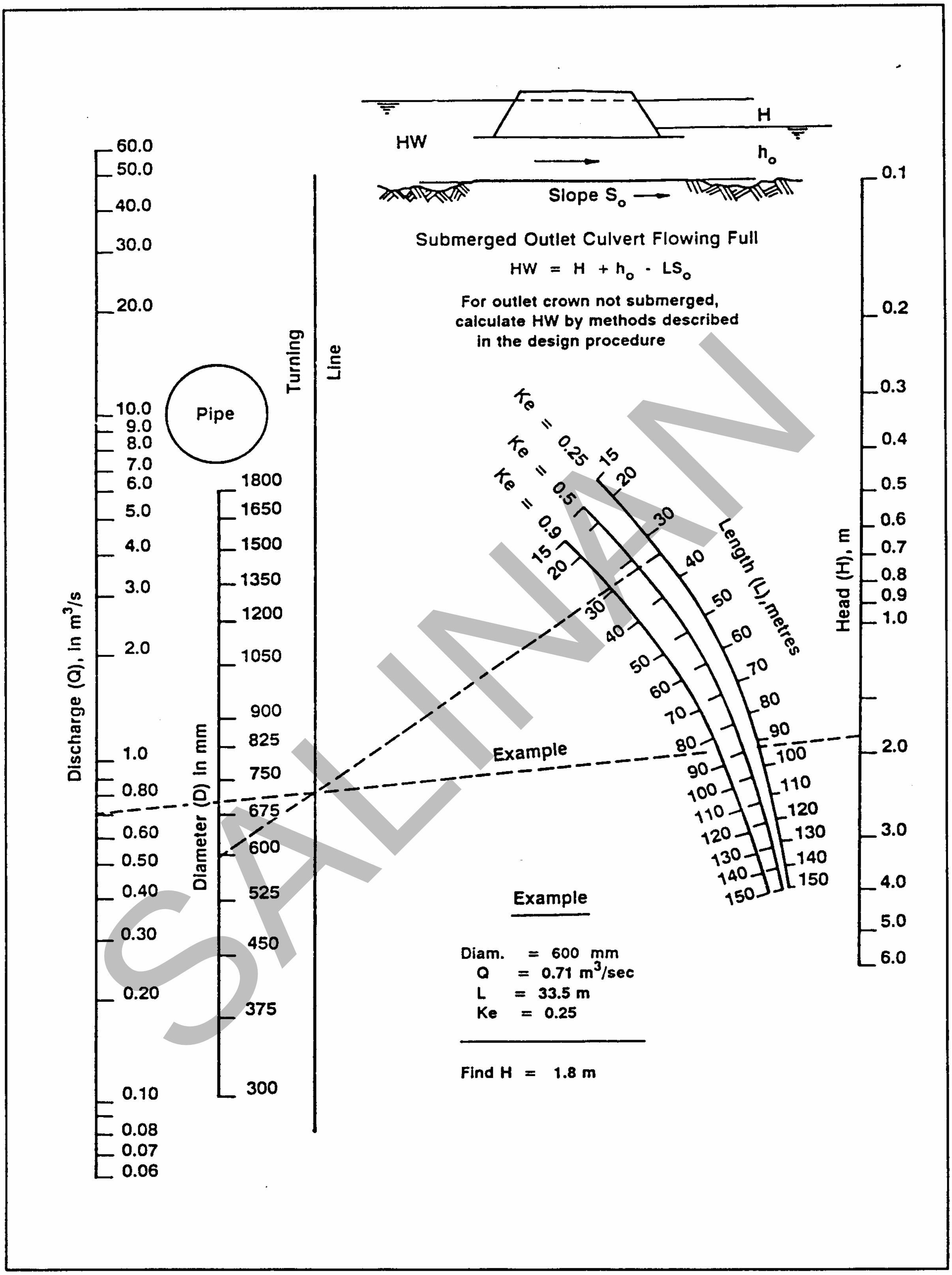

Figure 6.32 - Headwater Depth for Standard Corrugated Steel Culverts Flowing Full with Outlet Control n = 0.024

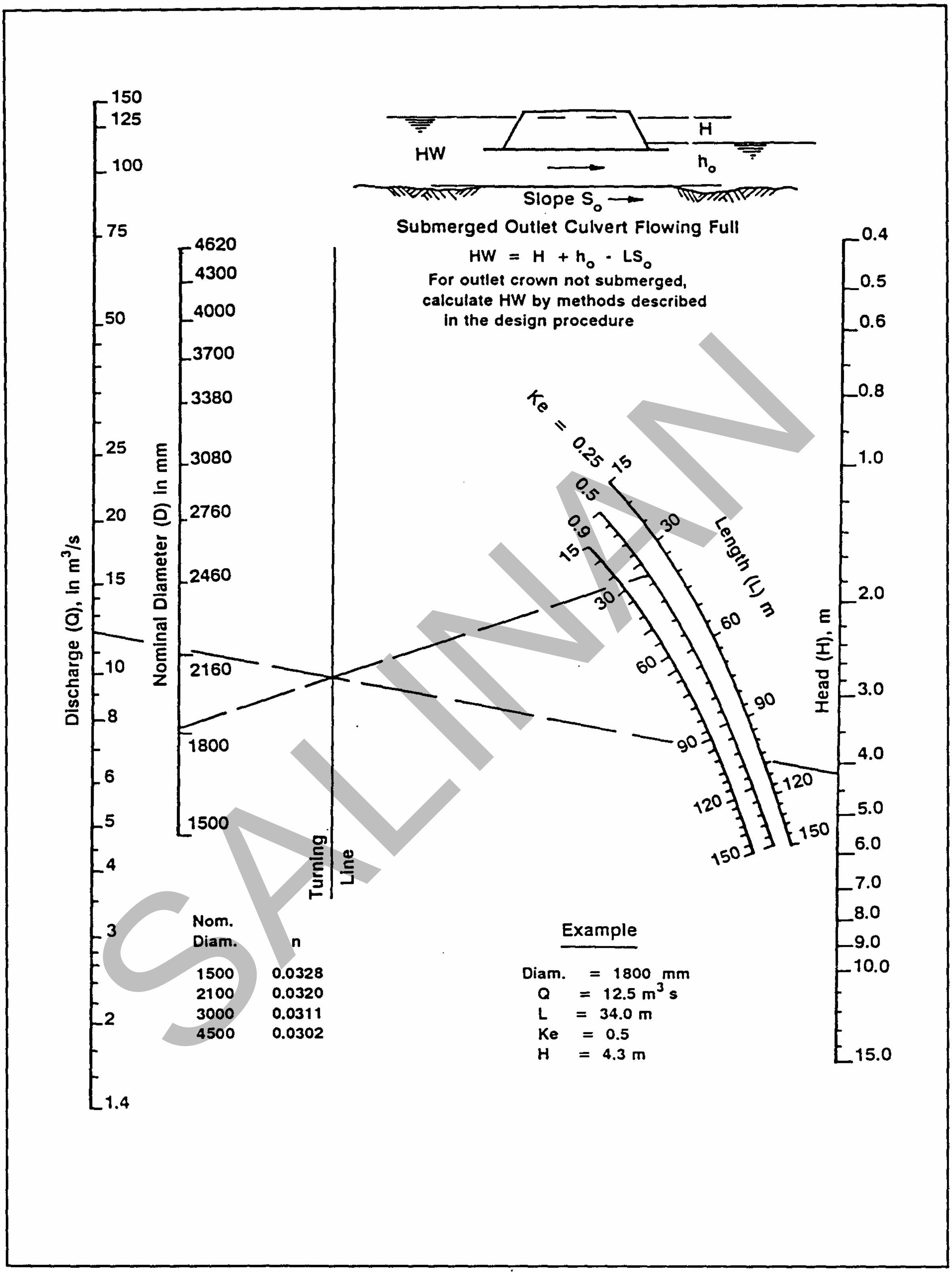

Figure 6.33 - Headwater Depth for Structural Steel Plate Corrugated Metal Pipe Culverts Flowing Full with n = 0.0328 to n = 0.0302

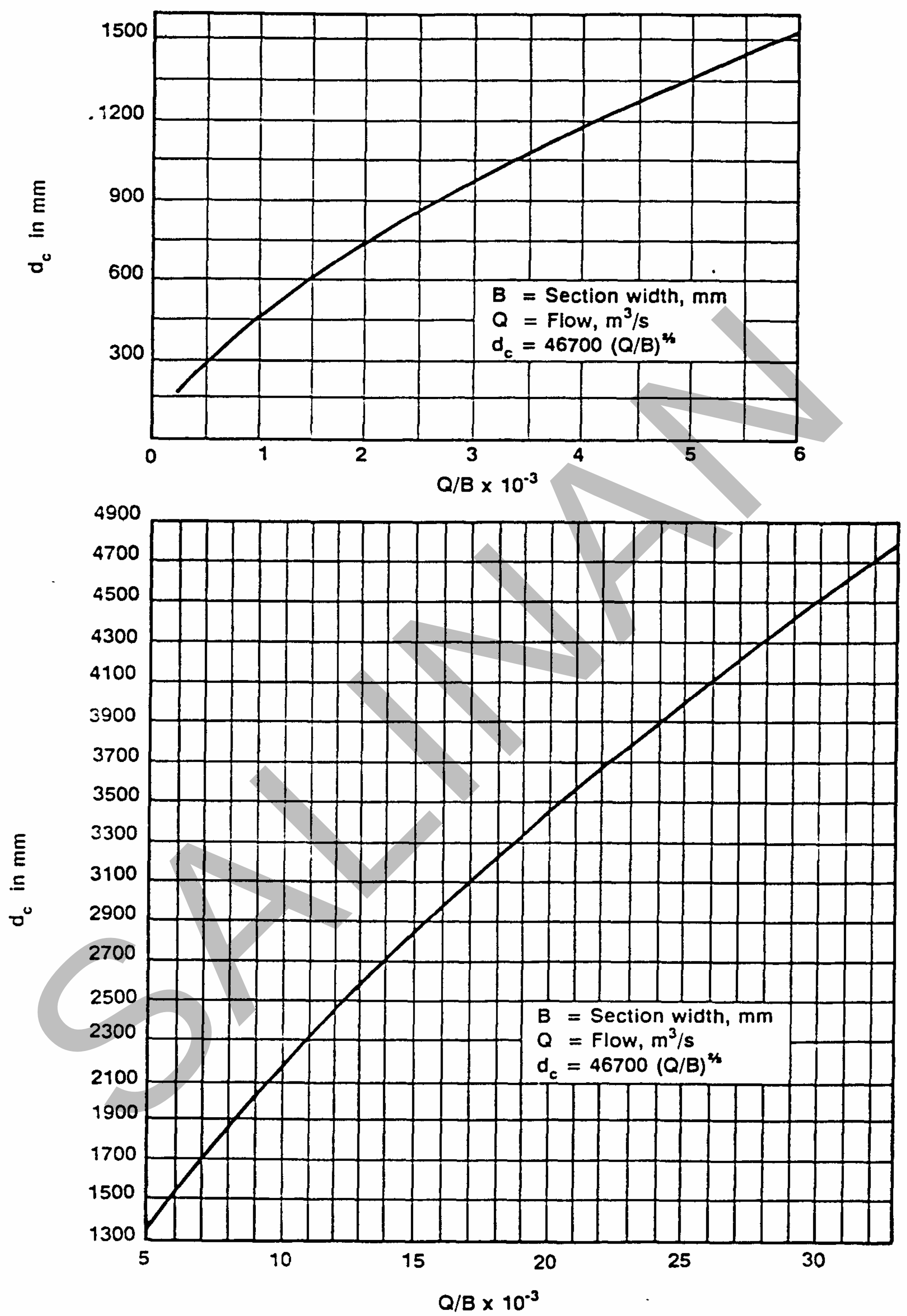

Figure 6.34 - Critical Depth d_c - Rectangular Section

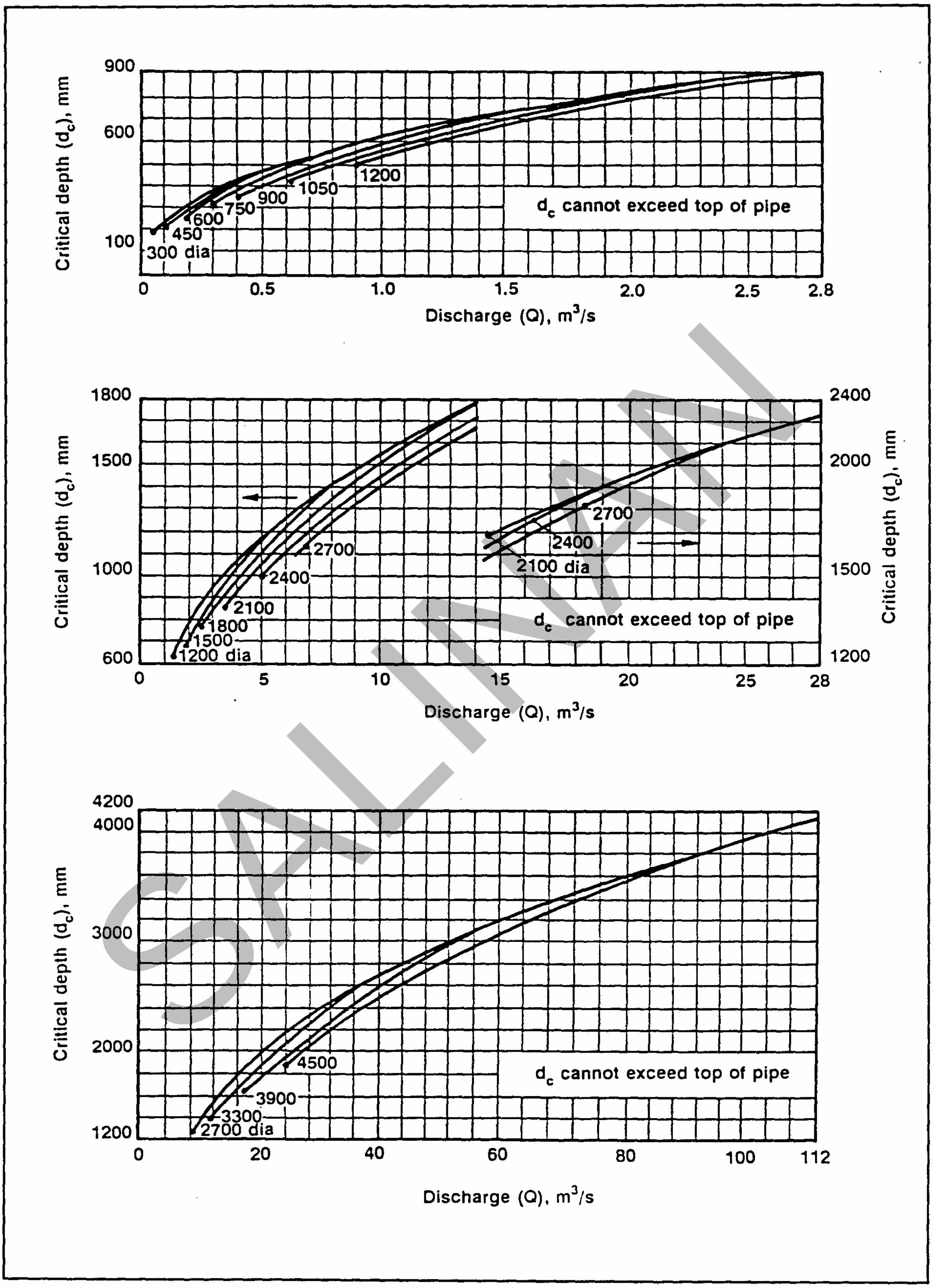

Figure 6.35 - Critical Depth d_c - Circular Pipe

6.5 FLOOD-CROSSING WATERWAY DESIGN

6.5.1 Scope

This section of the manual contain a brief discussion on the hydraulics of flood-crossings and downstream erosion protection measures for these types of crossings.

6.5.2 Introduction

A flood-crossing provides for flow across a road at a specific location, under conditions determined by the designer. Flood-crossings can be divided into two types :

- **Causeway**

  A roadway across a watercourse or across tidal water, especially constructed to resist the effects of submergence.

- **Floodway**

  A roadway across a shallow depression subject to flooding, especially constructed to resist the effects of submergence.

From these definitions a *floodway* is a special case of the *causeway* where approach velocities of flow can be expected to be low. In design there exists three problems :

a. design of a flood-crossing that will discharge the anticipated flow at an acceptable standard

b. design of a pavement to resist submergence and velocity of flow

c. design of protection to ensure the stability of the flood-crossing.

This section of the manual will detail the design for problems a. and c. given above.

6.5.3 Hydraulics

Where a flood-crossing is constructed at ground level and does not interfere with the flow, the depth of flow and velocity can be calculated using the slope-area method outlined in Section 6.2.

Where the flood-crossing is constructed above ground level, flow may be free or submerged. In the initial stages of overtopping a low tailwater condition usually exists, and free flow occurs. Under these conditions critical flow occurs at the crown of the road and the discharge is determined by the upstream head. At higher tailwater levels, when the depth of flow over the flood-crossing is everywhere greater than the critical depth, the discharge is controlled by the capacity of the downstream channel as well as the upstream head. Under conditions of tailwater control, the flow is described as submerged. The transition from free flow to submerged flow with rising tail water level is abrupt, and the flow pattern existing prior to the charge is described as incipient submergence.

Free flow may be further subdivided into plunging flow and surface flow. Plunging flow occurs when the streamlined flow penetrates the tailwater surface and produces a submerged hydraulic jump on the downstream slope. Surface flow occurs when the flow separates from the surface of the flood-crossing and overlays the downstream tailwater. The free flow transition is the range of tailwater levels within which a given discharge can produce either plunging flow or surface flow depending upon prior conditions. In general the plunging jet is of particular interest because of its more severe erosive effects.

Discharge over a flood-crossing can be determined using Figure 6.36 and the procedure outlined in Table 6.11.

Table 6.11 - Procedure for Determining Discharge for a Flood-Crossing

Step	Calculation Procedure
Step 1	Calculate ***H/l*** where H = $h + V^2/(2g)$ h = headwater above flood-crossing crest (m) V = average approach velocity (m/s) g = 9.81 m/s^2 l = width of flood-crossing (m)
Step 2	Enter Figure 6.36, Curve B, with ***H/l*** and obtain the free flow coefficient of discharge, C_f. Should the value of ***H/l*** be less than 0.15, C_f should be read from Curve A.
Step 3	If submergence is present (for example, if ***D/H > 0.7***) calculate percent submergence, ***(D/H) x 100***, where ***D*** = tailwater depth (m) and read off the submergence factor C_s/C_f.
Step 4	a. Calculate discharge over flood-crossing using the broad crested weir formula : $Q = C_f\, L\, H^{3/2} \times \dfrac{C_s}{C_f} \quad m^3/s$ (6.35) b. The tailwater level ***D*** (m) may be estimated from observation of flood debris and other evidence of high water marks on the banks of the stream or for a known ***Q***, it may be calculated using the *slope/area method* (see Section 6.2). c. For the free flow condition the critical velocity and critical depth can be calculated for the control section as indicated in Section 6.2 of this manual. For roads with various cross-sections the control section will be at the following points : • *crowned section* - at road crown • *downstream camber* - at upstream edge of pavement • *upstream camber* - at downstream edge of pavement

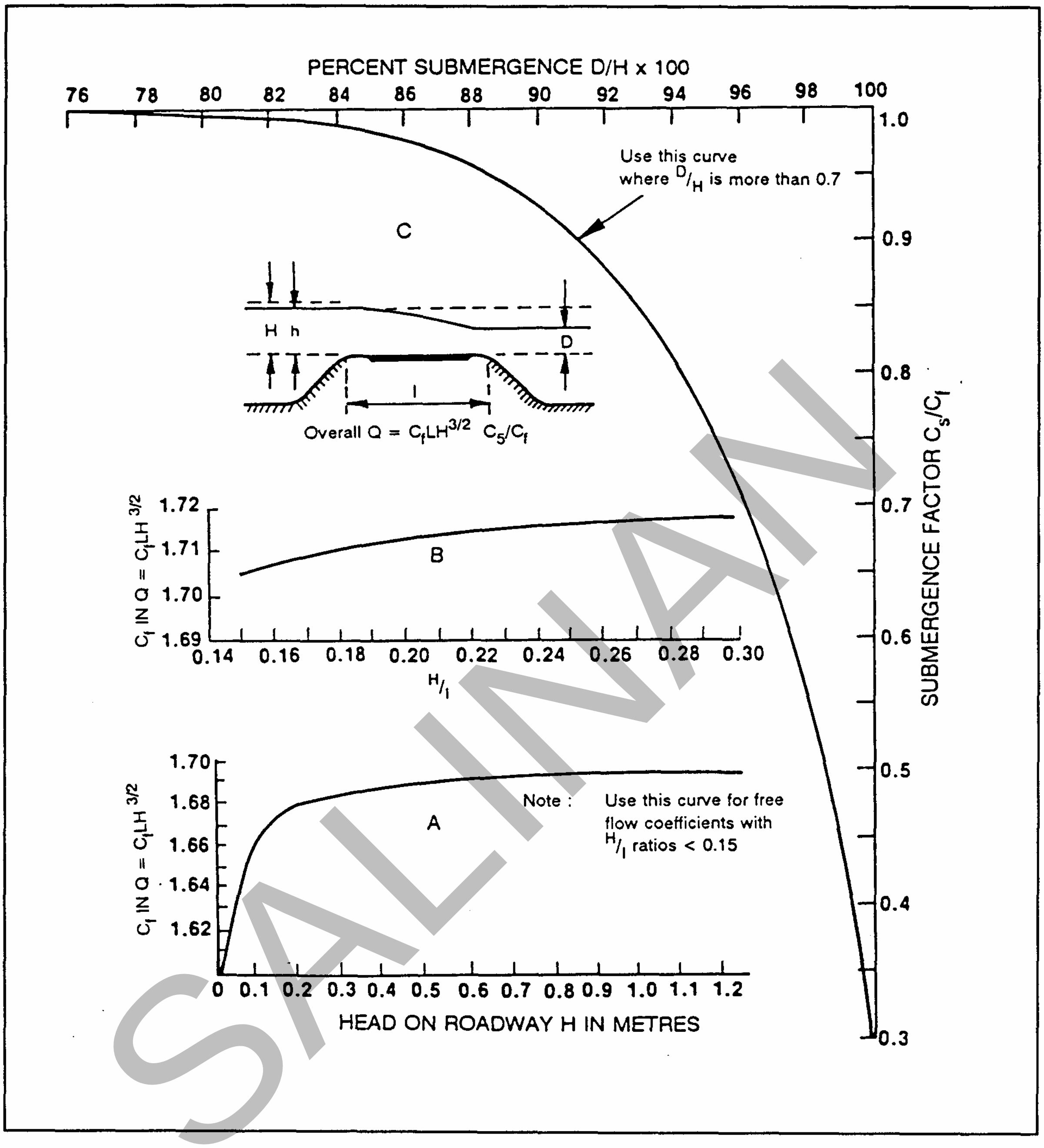

Figure 6.36 - Discharge Coefficients for Flow over Roadway Embankments

Table 6.12 - Worked Example - Flood-Crossing with Free Flow Condition

Step	Calculation Procedure
Detail	Crowned section $H = 0.30\ m$ $l = 7.4\ m$
Step 1	$H/l = 0.30/7.4 = 0.04$ which is < 0.15
Step 2	Enter Figure 6.36, Curve A, with $H = 0.30\ m$, $C_f = 1.68$ $q = C_f H^{3/2} = 1.68 \times 0.30^{3/2} = 0.276\ m^3/s$ /m length of crest
Step 3	From Section 6.2 $V_c = \sqrt{g\, d_c}$ $q = V_c\, d_c = g^{1/2}\, d_c^{3/2}$ then $d_c = \left[\dfrac{q}{g^{1/2}}\right]^{2/3} = \left[\dfrac{0.280}{9.81^{1/2}}\right]^{2/3} = 0.2\ m$ and $V_c = \dfrac{0.280}{0.2} = 1.4\ m/s$

6.5.4 Design Considerations

a. General

Flood-crossings are provided, generally, where traffic volumes are low, under the following circumstances :

- Where it is impractical or uneconomic to construct a bridge or culvert.
- Where flow across the road will be infrequent or of short duration.
- In conjunction with a bridge or culvert as a relief to take flows in excess of the flow for which the bridge has been designed.

b. Submergence and Trafficability

Extensive experiments have been conducted (Bonham and Hattersley, Reference 6.2) to ascertain the performance of motor vehicles negotiating flooded causeways. Buoyancy reduces the reaction between the tyres and the causeway surface, and at the same time the flow of water produces a lateral pressure against the side of the car. The car proceeds until the lateral pressure exceeds the maximum frictional resistance which can be developed by the car tyres under the reduced loading. Buoyancy effects are most severe upon the rear wheels as the fuel tank, sealed boot and other body compartments are located towards the rear. Consequently the rear wheels slide and probably spin and the car slews and faces upstream. The car probably then rolls backwards off the flood-crossing into deeper water.

Bonham and Hattersley (Reference 6.2) found that under ideal conditions cars operate with safety up to a depth of flow of 365 mm. However, they adopted a depth of 230 mm as the limit of trafficability, because in actual conditions waves and debris are present on the flood crossing. The depths of flow and associated maximum velocities found to be trafficable by Bonham and Hattersley (Reference 6.2) are listed in Table 6.13 .

Table 6.13 - Limits of Trafficability

Depth (mm)	Velocity of Flow (m/s)
300	1.13
250	1.51
200	1.85
150	2.48

Because cars have become lighter since Bonham and Hattersley (Reference 6.2) carried out their experiments. It is recommended that a 200 mm be adopted as the limit of trafficability, beyond which the flood-crossing is described as submerged. *Time of submergence* is then the time for which depth of flow over the flood-crossing exceeds 200 mm.

c. Vertical Alignment

The vertical alignment of a flood-crossing is determined by hydraulic adequacy, structural stability, safety, effect of backwater upon land use and design standards.

The inverts of flood-crossings should be kept level, so that motorists entering the flood-crossing, when it is flowing, are not confronted with concealed changes in depth of flow. An exception to this is a skew crossing of a major stream, where the natural stream grade must be estimated and applied in proportion to the causeway.

d. Cross-Section

From a study of flow patterns Bonham and Hattersley (Reference 6.2) concluded that causeways with downstream camber were to be preferred as they induced smooth, stable flow free from waves. However, if the causeway is subjected to submerged flow then a hydraulic jump may form on the causeway during the transition stage when the difference between upstream and downstream energy levels is small. The typical cross-section proposed by Bonham and Hattersley is shown in Figure 6.37 .

e. Drainage Under Road

Where flood-crossings are constructed in isolation, upstream drainage channels and culverts under the road should be provided to stop water, which may enter the road pavement and cause failure, from standing against the upstream side of the flood-crossing.

Where flood-crossings are constructed in conjunction with a bridge or culvert. The waterway under the road should be large enough to ensure that adequate tailwater is developed before the causeway is overtopped.

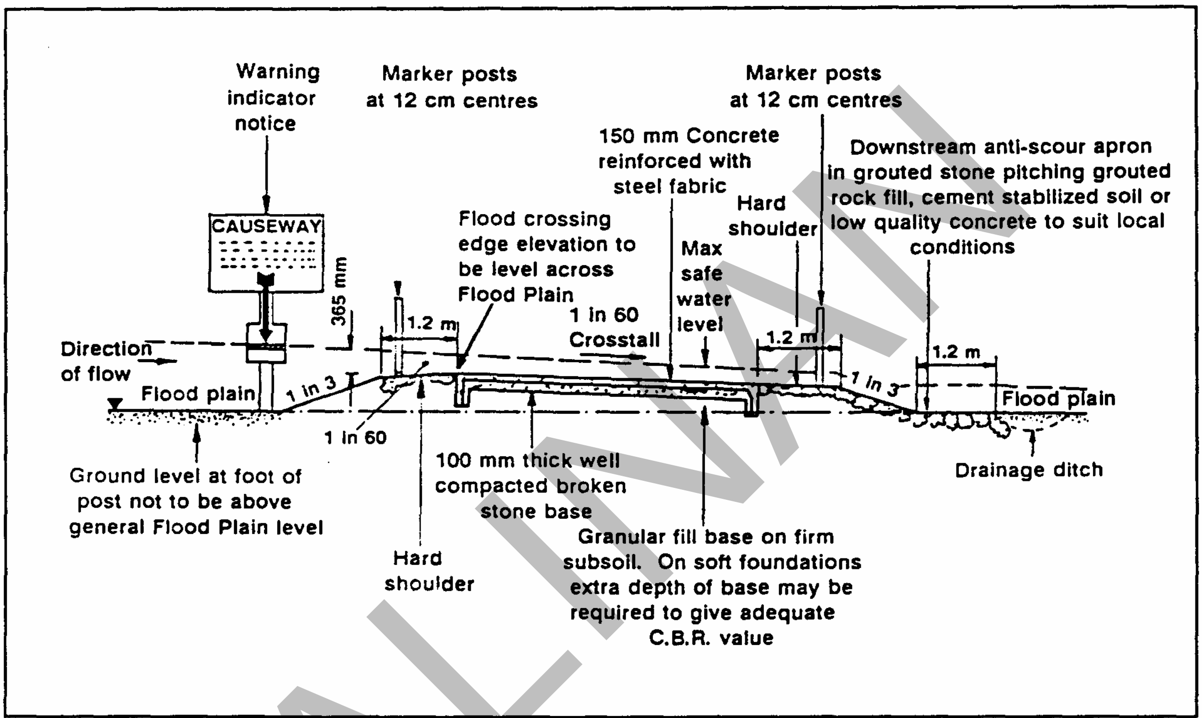

Figure 6.37 - Cross-Section of Typical Flood-Crossing

6.5.5 Protection

a. General

Where possible flood-crossings should be sited such that their height above ground level is kept to a minimum, hence downstream protection is also minimised.

Figure 6.38 (Reference 6.3) gives velocities of flow for a typical flood-crossing with free flow conditions.

Figure 6.38a shows plunging free flow with low tailwater, in which a high velocity jet passes down the downstream batter of the flood-crossing accelerating to a maximum velocity at the bottom of the batter. The sudden change of direction and bed friction decelerates the flow until the water level passes through the critical depth and a hydraulic jump occurs.

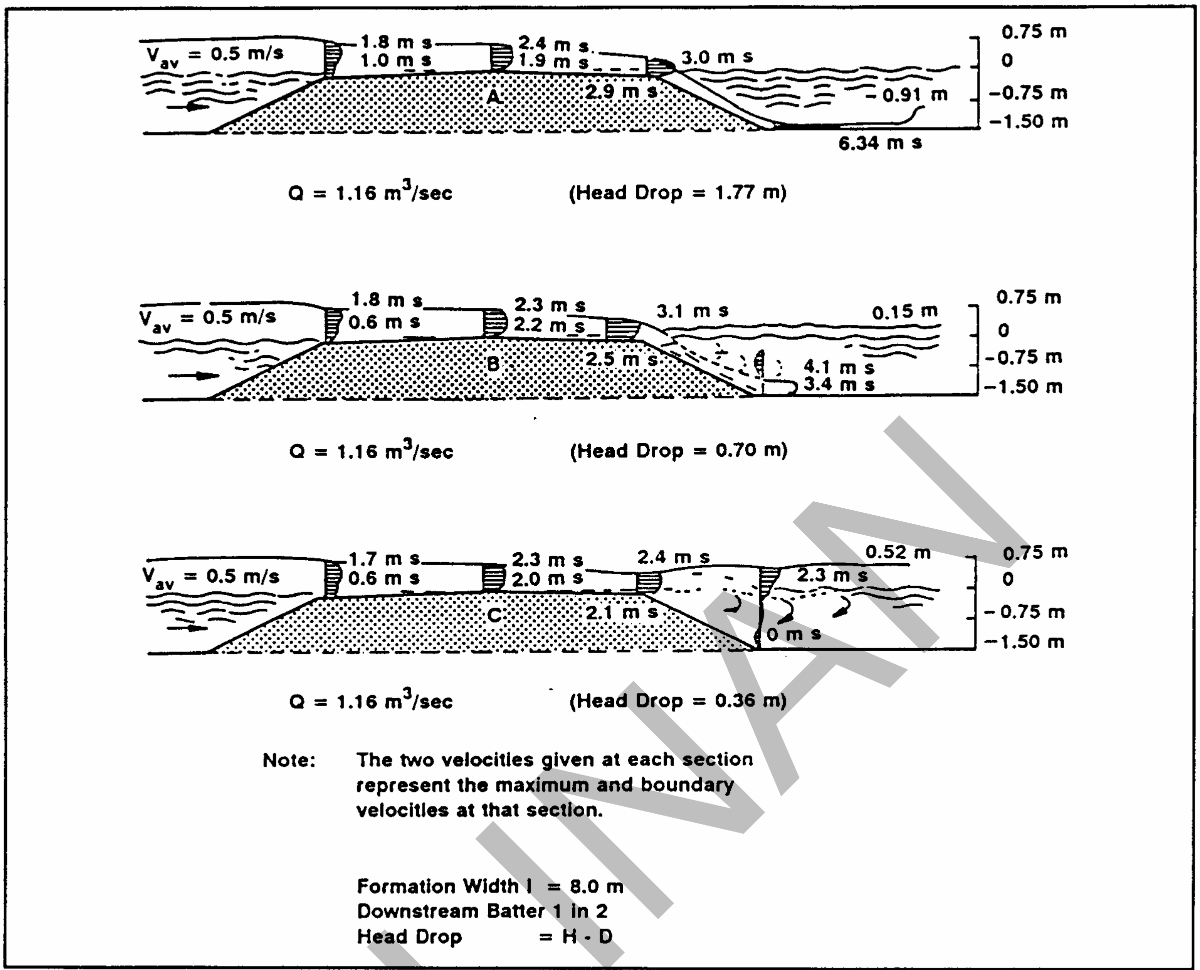

Figure 6.38 - Velocities Over a Typical Flood-Crossing

As the tailwater rises (Figure 6.38b) the hydraulic jump moves upstream until it reaches the downstream batter of the flood-crossing. At this stage free plunging flow occurs with the high velocity jet plunging into a turbulent body of water. The velocity of the jet reaches a maximum at or just below the surface of the tailwater and maintains this velocity down the batter and long the stream bed. Bed friction and eddy currents into the body of the tailwater gradually decelerate this jet.

With further rise in tailwater level submerged flow occurs (Figure 6.38c) when the total head drop across the embankment is slightly over 30 mm, the high velocity jet lifts from the boundary of the flood-crossing and stays on the surface of the tailwater. Eddies down into the tailwater gradually dissipate the energy of the jet in a harmless manner.

If condition *6.38c* can be assured only nominal batter protection is required. If, however, condition *6.38c* cannot be assured then either condition *6.38a* or *6.38b* will prevail and in either case protection will be required to the downstream batter and stream bed.

Downstream protection to flood-crossings may be either flexible or rigid (Reference 6.7). Examples of each type are as follows :

- **Flexible Protection**

  - *Dumped riprap* defined as graded stone dumped upon a prepared slope. In most areas dumped stone is the least costly type of protection.

  - *Wire-enclosed riprap* (Reference 6.7 and 6.8) is stone placed in wire baskets or in wire covered mats. Wire-enclosed riprap is generally used in locations where the only rock economically available is too small for dumped riprap.

- **Rigid Protection**

  - *Grouted riprap* is riprap with the interstices filled with portland cement mortar or weak concrete. It is generally used in locations where stone of a size suitable for other forms of riprap is not economically available.

  - *Concrete-slab riprap* is plain or reinforced concrete slabs poured or placed on the surface to be protected.

Hand-placed riprap which is inferior to dumped riprap (Reference 6.4) is not recommended for downstream protection works. Generally the use of flexible protection in the form of dumped riprap or wire-enclosed riprap is recommended for this purpose.

b. Dumped Riprap

Table 6.14 (Reference 6.5 and 6.8) gives details of the type and thickness of rock to be used as dumped riprap protection for downstream batters and aprons to flood-crossings. The recommended riprap type and rock is based upon 50% of the rock by weight being larger than the individual rock size required to resist the design bottom velocity of flow. The table is tentative and relies on the rock being well graded. Where the grading is poor and/or failure of the rock protection could lead to expensive maintenance a larger type of rock should be used.

Where necessary a filter should be placed between the embankment fill and rock protection. The filter may be a permeable plastic fabric membrane or graded sand/gravel filter.

Table 6.14 - Riprap Protection for Flood-Crossings

Flow Velocity (m/s)	**Recommended Riprap Protection**			
	Riprap Type	**Maximum Rock Diameter (m)**	**Approximate Rock Mass (kg)**	**Layer Thickness (m)**
up to 3.0	A	0.35	65	0.5
3.0 - 3.5	B	0.50	155	0.75
3.5 - 4.0	C	0.65	355	1.0
4.0 - 4.5	D	0.80	745	1.25
4.5 - 5.0	E	1.00	1450	1.6

Generally, where flood-crossings are designed for free flow an anti-scour rock apron about 1.2 m wide should be provided downstream from the flood-crossing batter.

6.6 REFERENCES

English Language References

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DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 7

SCOUR PREDICTION

FEBRUARY 1993

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7. SCOUR PREDICTION

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7. SCOUR PREDICTION

7.1 INTRODUCTION

Scour is defined as the displacement of stream bed or bank material by stream flow. This section of the manual covers the estimation of scour depths in the stream bed at a bridge site.

7.2 TYPES OF SCOUR

Scour can be divided into four different, but inter-related categories :

- *General Scour* which can occur naturally in the stream with or without a bridge crossing. It is a function of flow conditions and associated channel characteristics. It may occur over the full width of stream bed or be concentrated at bends.
- *Local Scour* which can occur because of the distortion of flow pattern in the immediate vicinity of the bridge piers and abutments.
- *Constriction Scour* which can occur generally at the bridge waterway because the flow is constricted by the bridge.
- *Degradation Scour* which is the lowering of the channel profile associated with geological processes or man-made regime changes.

Scour at a bridge may be caused by a combination of these categories.

Figure 7.1 (page 7-1) illustrates the common terms associated with scour at bridge waterways.

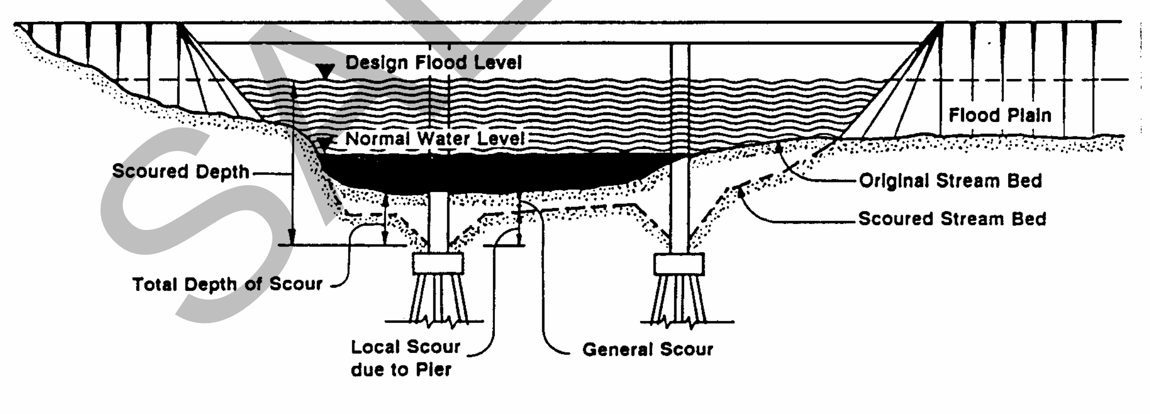


Figure 7.1 - Bridge Waterway Scour Terminology

7.3 FACTORS AFFECTING SCOUR

7.3.1 General

Local and constriction scour can be divided into two types :

- *The Clear Water Case* where material is removed from the scour hole and not replaced, and
- *The Sediment Transport Case* when the scour hole is continuously supplied with material from the sediment load carried on the stream bed; under these circumstances the stream bed is generally in motion.

It has been found experimentally (Reference 7.3) that the maximum depth of clear water scour is about 10% greater than the depth of scour for the sediment transporting case. Because the error involved in estimating the depth of scour is likely to be considerably greater than 10% the difference between the two types of scour is of no significance in the design situation.

Other factors which affect the depth of scour at a bridge site are :

- slope, and natural alignment of the channel
- potential for channel shift
- type and amount of bed material in transport
- history of flooding
- accumulation of debris
- constriction and/or realignment of flow due to the bridge and approaches
- layout and geometry of training works
- geometry and alignment of piers
- type and condition of bed material
- placement of rock or other protective materials
- natural or man-made changes in flow regimes
- vegetation in bed of ephemeral streams.

7.3.2 Constriction and/or Realignment of Flow

The embankment fills of a road crossing will often create a constriction of the stream in flood. The flood plain flow must then move laterally to the bridge opening. Where this lateral movement takes place is very important. If the flow returns to the channel in a reach of some length upstream from the bridge, there may be constriction scour over the entire waterway opening. If, however, lateral flow occurs along the embankment there may be local scour at the abutment, possibly extending out to the first or second pier. Constriction scour may also occur downstream from the structure and beneath it. Patterns of flow and scour effects at a particular site will depend upon the topography and vegetation at the site, and the waterway provided.

The flow will seek the easiest route, and the scour potential can only be assessed by first predicting the flow pattern for the conditions that will prevail during the life of the bridge.

7.3.3 Bed Material

The stream bed material plays an important part in the scour that can occur.

For non-cohesive materials the main resistance to erosion is provided by the submerged weight together with the size distribution of the particles.

For cohesive materials the resistance to erosion is controlled by the electrochemical bond between individual particles. Standard soil mechanics tests and index properties have not proved very satisfactory as a basis for erosion resistance criteria (Reference 7.9).

For weak sandstones and weakly cemented sand and gravels, it is important to determine whether the cementing medium will be dissolved during the life of the structure to the point where the material acts as if it is non-cohesive. Laminated materials such as hard shales may appear capable of withstanding high velocities, but in practice may tend to peel off during major floods.

7.4 METHODS OF ESTIMATING SCOUR

7.4.1 General

Although there are many methods (References 7.1, 7.7 & 7.8) for estimating scour, there is currently no general established theory from which the probable depth of scour at a bridge site can be estimated with any degree of confidence.

Before any methods of estimating scour are used, a check should be made into their background and the variables considered examined, to establish whether they can be applied to the bridge site under investigation. It should be noted that most of the available methods are related to non-cohesive materials only.

It is recommended that depths of scour be estimated using at least two of the available methods and the results compared. All estimates of depths of scour should be treated with caution and sound engineering judgement exercised in their application.

Section 7.4.2 (page 7-4) outlines a simple and conservative design approach for scour which involves limiting the flow velocity such that severe scouring will not occur. This method should be used as a first estimate of the bridge waterway area.

Sections 7.5 to 7.9 (pages 7-8 to 7-39) detail some of the more complex methods for estimating scour depths in the stream bed at a bridge site.

7.4.2 Simple Method for Scour Design

This method (Reference 7.15) involves the use of permissible flow velocities to limit scour in the stream bed. The intention of this simple approach is to limit *scour*, considered as being only the general movement of the stream bed due to excessive flow velocities, by limiting the stream velocity for a given discharge and flood height through provision of adequate water area (see Figure 7.2, page 7-4).

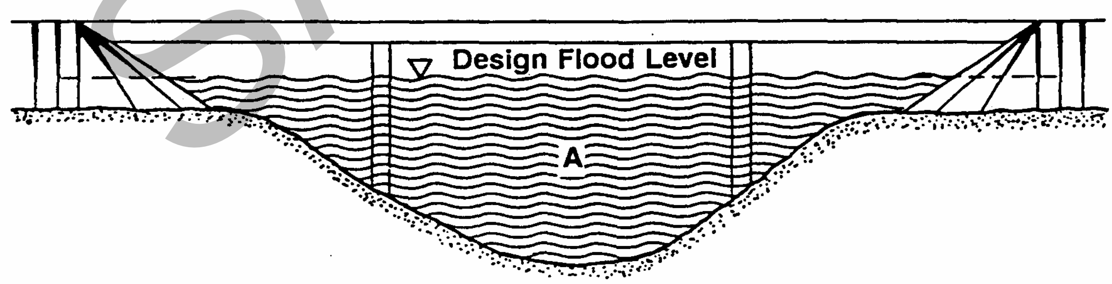


Figure 7.2 - Provision of Waterway to Limit Velocity

For a given discharge ***Q***, sufficient waterway area ***A*** is provided to limit the stream velocity ***V*** through the constricted bridge opening :

$$\frac{Q}{A} < V_{design} \quad (7.1)$$

The waterway area ***A*** is measured normal to the flood flow direction and excludes projected areas of piers.

The design velocities V_{design} to be adopted require a knowledge of flow velocities which will cause movement of different types of stream bed materials.

The approximate flow velocities which will just cause movement of the stream bed are shown in Table 7.1 (page 7-5).

Table 7.1 - Approximate Scouring Velocities

Stream Bed Material	Type	Scouring Velocity (m/s)
Silt	-	< 0.3
Sand	fine	< 0.3
	coarse	0.4 - 0.6
Gravel	6 mm	0.6 - 0.9
	25 mm	1.3 - 1.5
	100 mm	2.0 - 3.0
Clayey Soil	soft	0.3 - 0.6
	stiff	1.0 - 1.25
	hard	1.5 - 2.0
Stones	150 mm	2.5 - 3.0
	300 mm	4.0 - 5.0

The maximum permissible flow velocities adopted in design of bridge waterways to limit scour are shown in Table 7.2 (page 7-6). For economic reasons, these velocities are higher than those in Table 7.1 (page 7-5).

Table 7.2 - Approximate Maximum Permissible Design Flow Velocities

Stream Bed Material	Type	Maximum Permissible Design Flow Velocity (m/s)
Silt, sand		0.5
gravel	6 mm	1.5
Clay, firm loam	-	2.0
Gravel	100 mm	2.5
Stones	≥ 150 mm	3.5
Sound rock	-	4.5

7.4.3 Design Flood

Scour should be estimated for the maximum flood the bridge will pass without overtopping, as well as for the design flood.

7.4.4 Site Investigation

Sub-soil investigation in the form of bore holes, soil samples and testing is extremely useful in the estimation of scour.

The presence of underlying bedrock will give an absolute value of scour limit, but even a sudden change in the type or size of bed material with depth will indicate previous scour depths.

An indication of the scour potential of cohesive material can be obtained by soaking a sample in water and observing whether it shows indications of swelling or breaking down.

7.4.5 Safety Margins Against Scour

The equations provided in the following sections of this manual are considered to provide conservative estimates of scour. However, because of the inherent uncertainty of scour estimates and the complex considerations involved, it is difficult to give general guidance on safety margins against scour. Hence the following factors should be taken into account in the final analysis :

- long term trend in aggradation or degradation
- reliability of the basic data, especially hydrologic and geotechnical
- probability that extreme flows might exceed limits selected for design estimates
- seriousness of the consequence of total or partial failure of the protection measures
- experience of the designer in comparable situations
- additional cost of providing more security.

7.5 GENERAL SCOUR

General scour depth is defined as the depth to which the stream bed is scoured in the bridge waterway below the natural upstream bed level.

7.5.1 Method G1 - New Zealand Railways Method

Basis of Method

The method for estimating *general scour* detailed in this section is based upon that given in *Code of Practice for the Design of Bridge Waterways* produced by the New Zealand Ministry of Works and Development (Reference 7.11).

Estimation Method

The New Zealand Railways have developed a method of scour estimation based on investigation into past scour failures at a considerable number of railway bridges. The method, proposed by P.S. Holmes (Reference 7.10), considers *general scour* (described in this section) as well as *local scour* (see Section 7.6.1, page 7-12). The depth of *general scour* (m) is given by :

$$D_{s1} = \frac{Y_r\, V_o\, K}{\sqrt{A/W}} \quad \text{or} \quad Y_o \quad \text{whichever is greater} \tag{7.2}$$

$$V_o = \frac{Q}{A}\left[\frac{Y_0}{A/W}\right]^{2/3} \times C \tag{7.3}$$

$$K = \sqrt{\frac{W}{4.83\, Q^{1/2}}} \quad \text{but} \not> 1.0 \tag{7.4}$$

where

D_{s1} = general scour depth (m), that is, depth from water surface to stream bed level after scour has occurred

Y_o = maximum flood water depth in line with, and upstream of, zone of scour (m)

Y_r = rise in water level to flood level immediately upstream of bridge site measured from normal low water level (m)

V_o = mean velocity of flow in vertical section upstream of zone of scour (m/s)

K = a modify factor dependent on ratio of waterway width and Lacey regime width

A	=	waterway area at the bridge site, normal to flow, using unscoured bed profile, with no reduction being made for projected area of piers (m^2)
W	=	total waterway width at the bridge site (m)
Q	=	peak flood discharge at bridge site (m^3/s)
C	=	***1.2*** where converging flows are encountered
	=	***1.0*** for other cases.

7.5.2 Method G2 - Method from C.R. Neill

Basis of Method

The method for estimating *general scour* detailed in this section is based upon *Guide to Bridge Hydraulics* edited by C.R. Neill (Reference 7.2).

Estimation Method

The depth of general scour in unrestricted alluvial streams may be estimated by using Lacey's empirical regime formula (References 7.2, 7.4 & 7.5).

$$d_m = 0.5 \left[\frac{Q}{f}\right]^{1/3} \qquad (7.5)$$

$$f = 1.76\sqrt{m}$$

where d_m = mean depth (m) of scour measured from water surface at design discharge.

Q	=	flow in main channel (m^3/s)
f	=	Lacey's silt factor (see Table 7.3, page 7-10).
m	=	grain size (mm)

Where the flow is in a well defined channel ***Q*** is taken as the total design discharge, but where there is significant floodplain flow outside the main channel ***Q*** is taken as the flow in the main channel only.

Table 7.3 - Values of Lacey's Silt Factor f

d_{50} = median diameter (mm) of bed sand by weight	Value of Lacey's Silt Factor *f*
0.06	0.4
0.1	0.6
0.2	0.8
0.3	1.0
0.5	1.2
0.7	1.5
1.0	1.8
1.3	2.0

NOTES

1. *d_{50} is found by sieve analysis and is the size exceeded by **50%** of the bed sample.*

2. *Unless experience indicates otherwise, **f** should normally be taken as **1.0** for sandy materials.*

Equation (7.5) (page 7-9) gives the estimated mean scour depth across the channel section. To estimate the maximum natural scoured depth, d_m is multiplied by a factor c given in Table 7.4 (page 7-11). That is the maximum natural scoured depth

$$d_{max} = c\, d_m \qquad (7.6)$$

where c = multiplying factor

d_m = mean depth (m) of scour measured from water surface at design discharge.

Table 7.4 - Multiplying Factors for Maximum Scoured Depth

Natural Section	Multiplying Factor c
Straight reach	1.25
Moderate bend	1.50
Severe bend	1.75
Right-angled bend	2.00

It should be noted that in some streams the observed depth of scour has been up to twice that calculated.

The *competent velocity method* given in Section 7.7.2.b (page 7-30) can be used as a rough check on general scour.

7.6 LOCAL SCOUR

Local scour is defined as the lowering of the stream bed adjacent to structures (such as bridge piers, groynes and abutments) below the *general scour* level.

7.6.1 Method L1 - New Zealand Railways Method

Basis of Method

The method for estimating *local scour* detailed in this section is based upon that given in *Code of Practice for the Design of Bridge Waterways* produced by the New Zealand Ministry of Works and Development (Reference 7.11).

Estimation Method

The New Zealand Railways have developed a method of scour estimation based on investigation into past scour failures at a considerable number of railway bridges. The method, proposed by P.S. Holmes (Reference 7.10), considers *general scour* (see Section 7.5.1, page 7-8) as well as *local scour* (described in this section). The depth of *local scour* (m) is given by :

$$d_{s2} = 0.8\sqrt{V_0\, b} \qquad \textbf{(7.7)}$$

$$V_o = \frac{Q}{A}\left[\frac{Y_o}{A/W}\right]^{2/3} \times C \qquad \textbf{(7.8)}$$

where

d_{s2}	=	depth of local scour (m), that is depth from mean stream bed level
V_o	=	mean velocity of flow in vertical section upstream of zone of scour (m/s)
b	=	effective projected pier width (m)
Y_o	=	maximum flood water depth in line with, and upstream of, zone of scour (m)
A	=	waterway area at the bridge site, normal to flow, using unscoured bed profile, with no reduction being made for projected area of piers (m^2)
W	=	total waterway width at the bridge site (m)
Q	=	peak flood discharge at bridge site (m^3/s)
C	=	***1.2*** where converging flows are encountered
	=	***1.0*** for other cases.

7.6.2 Method L2 - Method from C.R. Neill

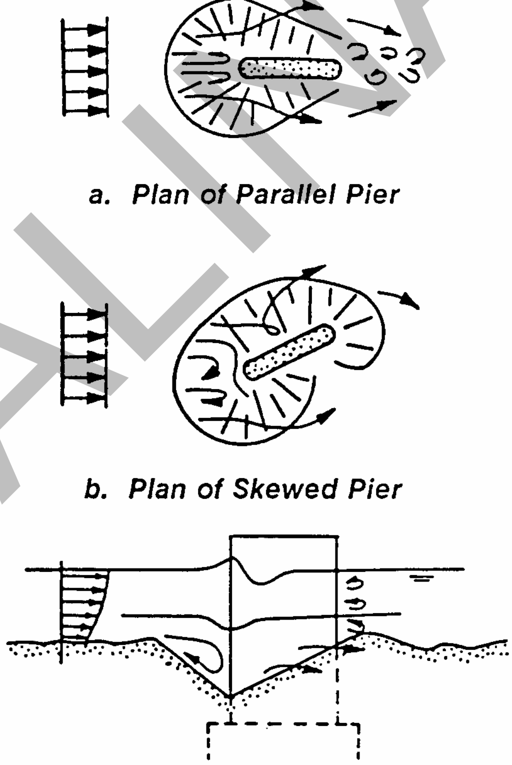
Basis of Method

The method for estimating *local scour* detailed in this section is based upon *Guide to Bridge Hydraulics* edited by C.R. Neill (Reference 7.2).

Estimation Method

Piers placed in waterway openings tend to produce local scour (Figure 7.3, page 7-13) even where they do not produce any significant reduction in the net waterway width. This local scour should be added to the *general scour* or *constriction scour* estimated as outlined in Sections 7.5.2 and 7.7.2 (page 7-9 and 7-29).

In general the depth of local scour depends on the pier width, length, shape and alignment, on footing details, on velocities and depths of flow, on the type and size of bed material, on the rate of bed transport and on accumulation of debris.



a. Plan of Parallel Pier

b. Plan of Skewed Pier

c. Elevation of Pier

Figure 7.3 - Usual Form of Local Scour Holes at Piers

a. Local Scour at Piers

Circular or Elongated Piers

The local depth of scour below the surrounding bed at the nose of a circular pier or elongated pier aligned parallel to the flow should normally be taken as equal to the effective pier diameter or width near general bed level multiplied by a *Local Scour Coefficient* C_L as given in Table 7.5 (page 7-15). These coefficients are intended as design values for non-cohesive materials that are expected to be mobile under design flow conditions, where no special protection is provided. Smaller coefficients may be appropriate in more scour resistant materials.

Effect of Pier Skew

If an elongated pier is substantially skewed to the direction of flow, local depths of scour may be very much greater than given by Table 7.5 (page 7-15). Skew angles θ greater than **5°** to ***10°*** should be avoided wherever practicable.

To estimate the effect of pier skew on local scour depth, the *Local Scour Coefficient* C_L is multiplied by the *Skew Coefficient* C_s given in Table 7.6 (page 7-16). Table 7.6 is based approximately on data given by Laursen (Reference 7.6) and other experimenters and is intended to indicate the approximate range of increase in local scour due to pier skew. The figures should be treated with caution, because there are substantial discrepancies between the results of different experimenters.

Effect of Debris

Accumulations of debris may substantially increase local pier scour. General experience is that a pier nose, semicircular in plan, and vertical or very slightly raked, is best for discouraging accumulation of debris.

For relatively slender piers, where the expected local scour would otherwise be small, some allowance should normally be made for an increase in effective width w due to accumulation of debris.

Depth of Local Scour

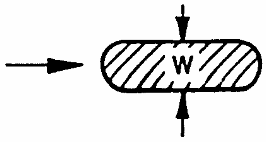
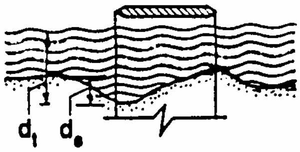
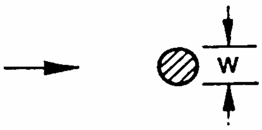
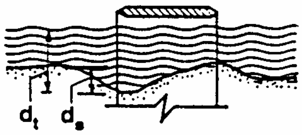
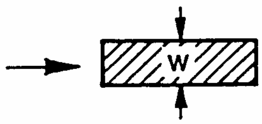
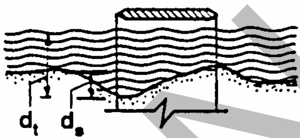
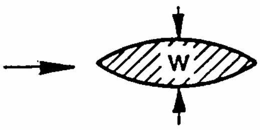
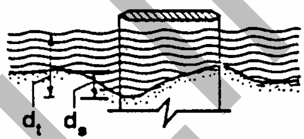
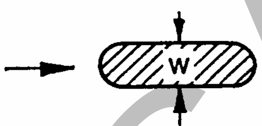
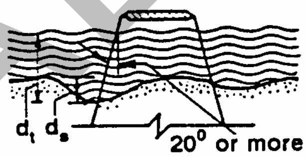
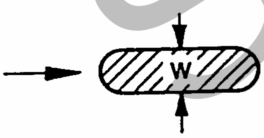
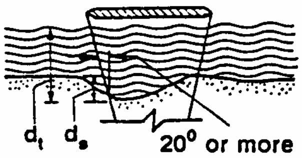
The depth of local scour at a pier in non-cohesive silt or sand can therefore be calculated as follows :

$$d_s = C_L\, C_S\, w \qquad \textbf{(7.9)}$$

where

d_s	=	depth of local scour (m), that is depth from mean stream bed level
C_L	=	*local scour coefficient* from Table 7.5 (page 7-15)
C_S	=	*skew coefficient* from Table 7.6 (page 7-16)
w	=	effective pier diameter or width (m) near mean stream bed level

Table 7.5 - Local Scour Coefficient C_L for Piers in Non-Cohesive Silt and Sand and Aligned Parallel to the Flow

Pier Shape in Plan	Pier Shape in Profile	Local Scour Coefficient C_L	
		$d_t \le 5w$	$d_t > 5w$
		1.5	2.3
		1.5	2.3
		2.0	3.0
		1.2	1.8
		1.0	1.5
		2.0	3.0

NOTE

*Where the depth of scour is likely to expose the footing below the flow, **w** shall be taken as the width of the footing.*

Table 7.6 - Skew Coefficient C_s for Piers Skewed at Angle θ to the Flow

Skew Angle θ	Length to Width Ratio of Pier in Plan		
	4	8	12
0°	1.0	1.0	1.0
15°	1.5	2.0	2.5
30°	2.0	2.5	3.5
45°	2.5	3.5	4.5

NOTES

1. *This table is intended to indicate approximate range only.*
2. *Design depths of scour for severely skewed piers, where the use of these is unavoidable, should preferably be determined by means of special model tests.*

b. Local Scour at Abutments

Reliable guidance on the estimation of local scour at abutments cannot be given because of the wide variation in geometry and approach flow conditions that can occur in practice, and because of a lack of experimental data.

Embankments projecting into wide flood plains may produce scour problems in two ways. First, the flow patterns of flood waters create concentration of flow at the upstream corners of the embankment. In many cases this results in a serious scour potential at the abutment. Second, the embankment constricts the waterway opening, with a corresponding increase in flow, influencing scour at piers near the abutment.

Model studies have yielded an insight into the scour produced by embankment protrusions. Figure 7.4 (page 7-17) shows the scour configuration expected for a normal embankment. Figure 7.5 (page 7-17) shows the influence of embankment on scour at adjacent piers. Figure 7.4 and Figure 7.5 merely indicate qualitative scour patterns.

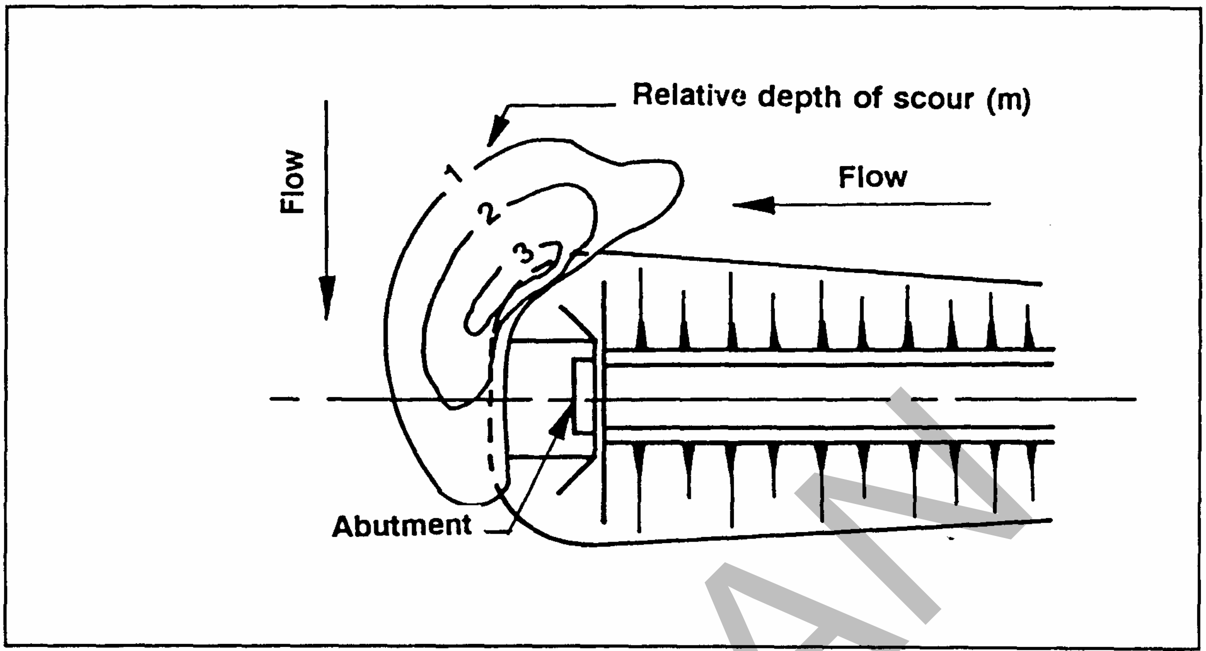


Figure 7.4 - Scour at an Embankment

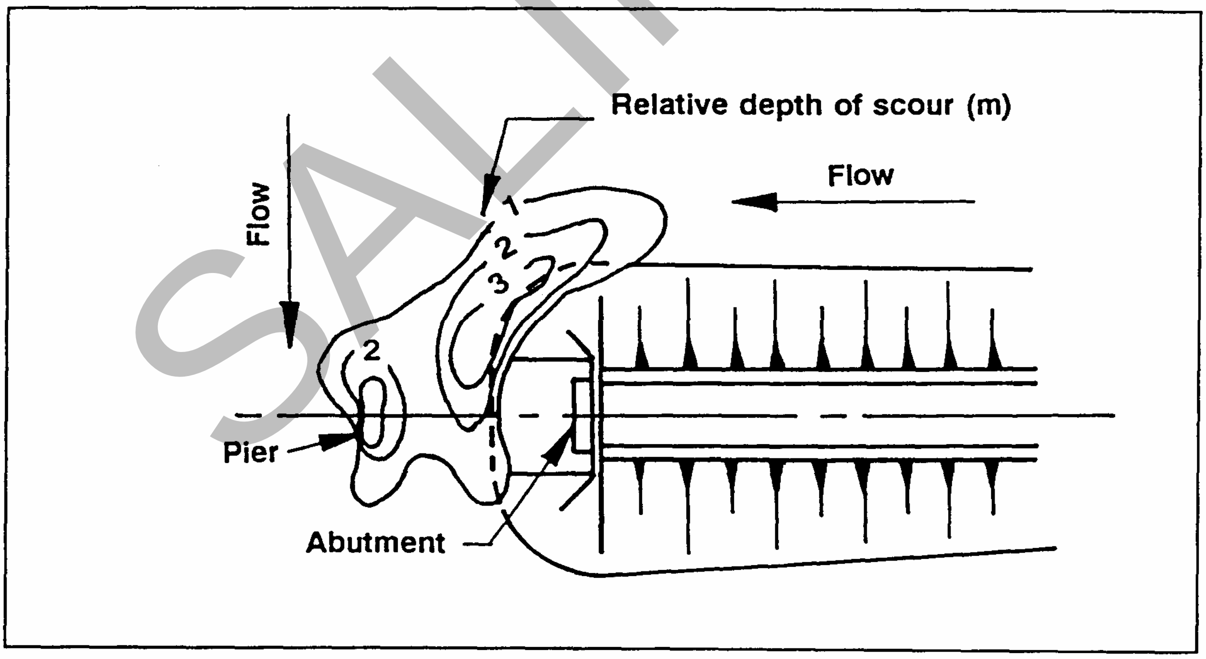


Figure 7.5 - Scour at an Embankment and Adjacent Pier

7.6.3 Method L3 - Method from Faraday & Charlton

Basis of Method

The method for estimating *local scour* detailed in this section is based upon *Hydraulic Factors in Bridge Design* by R.V. Faraday and F.G. Charlton (Reference 7.12).

Estimation Method

The methods of assessing local scour at piers, abutments and training works are detailed in the following sections.

a. Local Scour at Bridge Piers

i. Mechanism of Scour

Local scour around piers is the result of vortex systems which develop as the stream flow is deflected around the pier. The main vortex system which contributes to the formation of scour holes originates at the upstream nose of the pier where the flow acquires a downward or diving component in elevation, which reverses direction in plan at the stream bed. As bed material is removed by the flow a spiral roller develops within the hole formed, which spirals around the side of the pier. In plan, the vortex system has a horseshoe shape and is frequently referred to as a horseshoe vortex (see Figure 7.6, page 7-18).

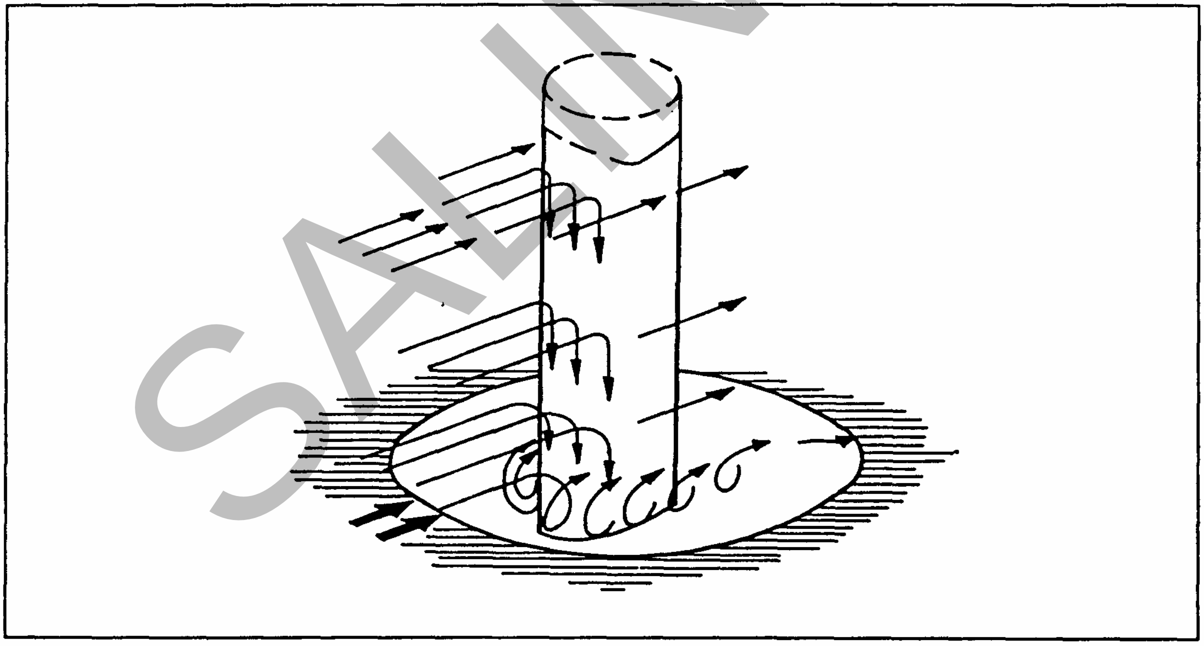


Figure 7.6 - Horseshoe Vortex Formation at a Cylindrical Pier

The scour hole will increase in size until an equilibrium depth is reached. The equilibrium depth is dependent on which of the following scour conditions prevail :

- *clear water scour*, where the bed movement occurs only adjacent to the piers. The equilibrium depth is reached when the shear stresses at the surface of the scour hole are insufficient to eject the particles
- *sediment transporting scour*, where the whole stream bed is in motion. The equilibrium depth is reached when the amount of sediment entering the scour hole is balanced by the amount leaving.

The depth at which the equilibrium condition is reached will be greatest at the transition between the clear water and sediment transporting conditions, that is, at the threshold of movement, when the approach velocity equals U_c, the average critical velocity for initiating sediment movement (see Figure 7.7, page 7-19).

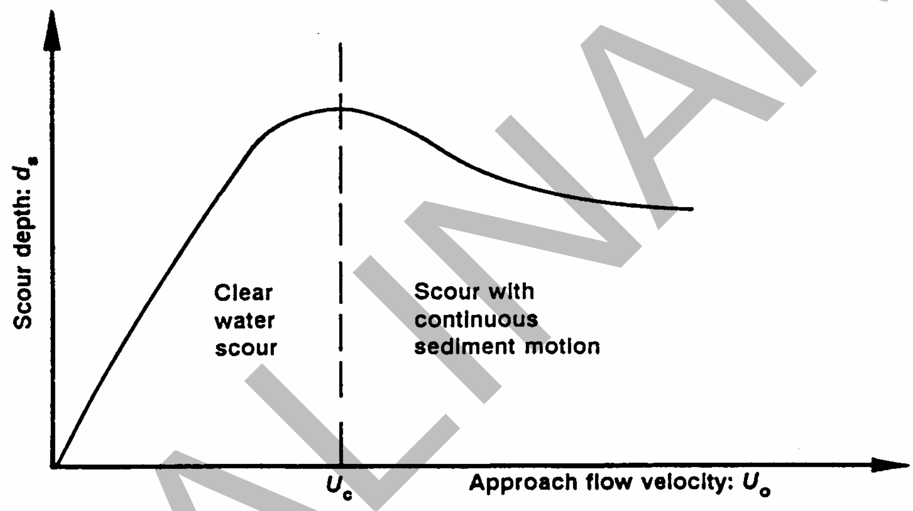


Figure 7.7 - Scour Depth versus Approach Velocity

To determine whether clear water or sediment transporting conditions prevail refer to Figure 7.8 (page 7-19) which shows the relationship between the particle entrainment function and particle Reynolds number. Clear water conditions are defined by points lying below the line and sediment transporting conditions by points above. The line represents the threshold of movement.

The entrainment function F_E is given by :

$$F_E = \frac{R\ S}{(s-1)\ D} \qquad (7.10)$$

For wide channels, this relationship approximates to :

$$F_E = \frac{y\ S}{(s-1)\ D} \qquad (7.11)$$

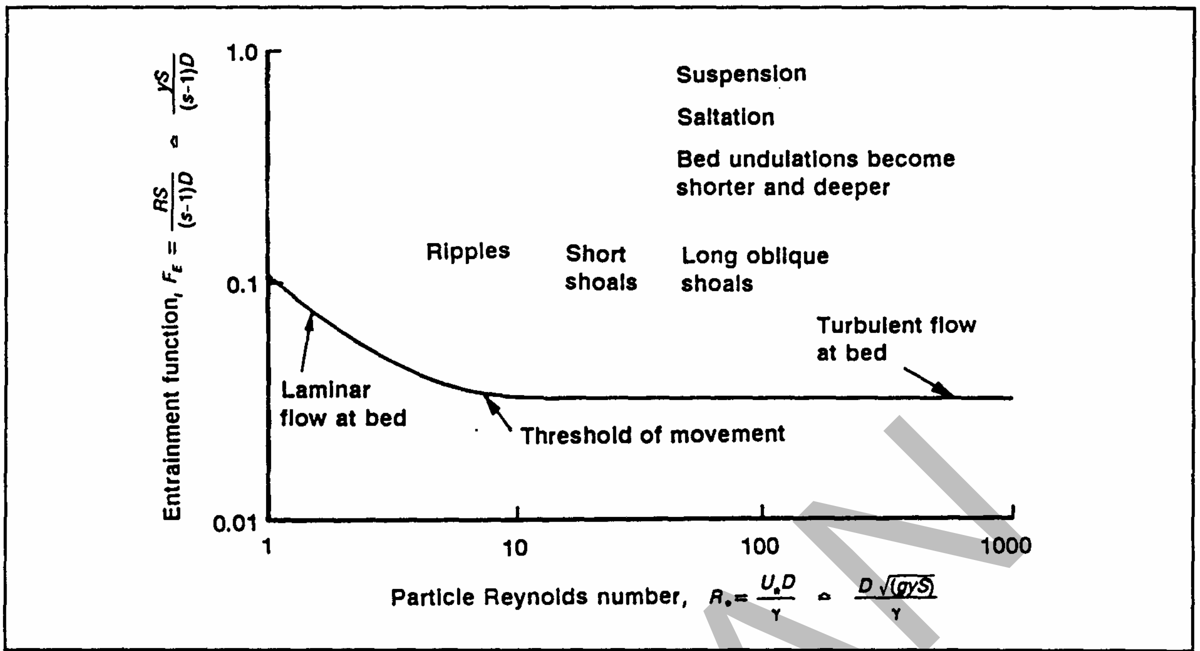


Figure 7.8 - Threshold of Bed Particle Motion

The particle Reynolds number, R_*, is given by :

$$R_* = \frac{U_* D}{\nu} \tag{7.12}$$

which, for wide channels, approximates to :

$$R_* = \frac{D\sqrt{g\,y\,S}}{\nu} \tag{7.13}$$

where
R = hydraulic radius (m) = A/P
S = hydraulic gradient
s = specific gravity of stream bed material
D = characteristic bed particle size (m)
A = flow area of cross-section (m^2)
P = wetted perimeter of flow area (m)
y = mean depth of flow (m)
U_* = shear velocity (m/s)
ν = kinematic viscosity (m^2/s)
= $1.14 \times 10^{-6}\ m^2/s$ for water at $15°C$

g = acceleration due to gravity (***9.81 m/s²***)

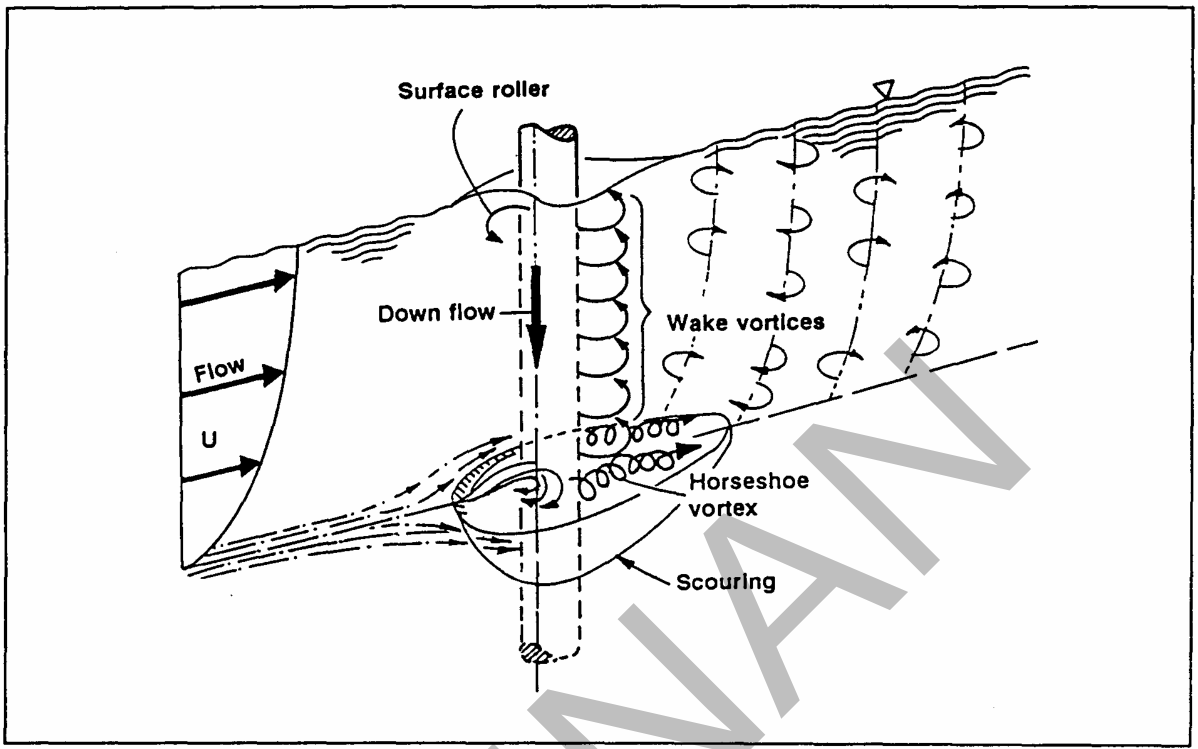


Figure 7.9 - Flow Pattern at a Cylindrical Pier

ii. Cylindrical Piers

The interaction of the flow around a bridge pier and the river bed surrounding it is very complex. Scour estimation entails using empirical methods or a combination of analytical and empirical techniques to fit equations for scour prediction to the available experimental and field data. Research has mostly concentrated on scour effects in non-cohesive material and there is little guidance available for estimating scour depths in cohesive materials.

Non-Cohesive Material

Table 7.7 (page 7-22) summarises the equations for estimating the depth of scour in non-cohesive materials adjacent to a cylindrical pier.

Table 7.7 - Estimation of Local Scour at Cylindrical Piers in Non-Cohesive Material

Stream Bed	Scour Condition	Froude Number $F=U/\sqrt{(gy)}$	Equation
Sand	Clear water	-	$d_s = 1.17\ U_0^{0.62}\ b^{0.62}$
	Sediment transporting	$F < 0.5$	$d_s = 1.11\ y_0^{0.5}\ b^{0.5}$
		$F > 0.5$	The larger value given by $d_s = 1.59\ U_0^{0.67}\ b^{0.67}$ *or* $d_s = 1.11\ y_0^{0.5}\ b^{0.5}$
		$F < 0.3$	$0.001 < D_{50} < 0.004$ $d_s = 1.8\ y_0^{0.75}\ b^{0.25} - y_0$ *or* $d_s = C\ y_0$ *and* $y_0 = 0.38\ q_0^{0.67}\ D_{50}^{-0.17}$
Gravel	Clear water	-	$d_s = C\ y_0$ $y_0 = 0.23\ (s-1)^{-0.43}\ q_0^{0.86}\ D_{90}^{-0.}$
	Sediment transporting	-	$d_s = C\ y_0$ $y_0 = 0.47\ q_0^{0.8}\ D_{90}^{-0.12}$

NOTES

*In cases where the Froude number exceeds **0.8**, a model investigation to determine scour effects is recommended.*

where d_s = depth of scour measured below upstream bed level (m)

b = width of pier (m)

U_o = approach flow velocity (m/s)

y_o = depth upstream of pier (m)

q_o = discharge per unit width upstream of pier (m^3/s)

D_{50} = median particle size of bed material (m)

D_{90} = size of bed material such that 90% of the particles by number are smaller (m)

s = specific gravity of stream bed material

C = coefficient $0.5 < C < 1.0$

Cohesive Material

There are very few reference data on scour in cohesive soils or on the effect of the degree on consolidation on resistance scour. It is therefore recommended that the simple formulae in Table 7.8 (page 7-24) based on pier width be used to estimate scour in cohesive soils.

Table 7.8 - Depth of Scour at Piers in Cohesive Soils

Pier Shape in Plan	Inclination of Pier Faces	Depth of Scour
Circular	Vertical	***1.5 b***
Rectangular	Vertical	***2.0 b***
Lenticular	Vertical	***1.2 b***
Rectangle with semi-circular noses	Vertical	***1.5 b***
	Inclined inward towards top : angle more than 20° to vertical	***1.0 b***
	Inclined outward towards top : angle more than 20° to vertical	***2.0 b***
NOTES *where **b** = pier width*		

iii. Non-Cylindrical Piers

Estimation of *local scour* at non-cylindrical piers may be obtained by applying suitable factors to the equations for predicting scour around cylindrical piers given in Table 7.7 (page 7-22).

Non-cylindrical piers may be designed to present a sharper nose to the oncoming flow than cylindrical piers. This has the effect of reducing the strength of the horseshoe vortex and hence the depth of scour. For piers designed with blunter noses the converse is true. Factors for adjusting for non-cylindrical shapes are given in Table 7.9 (page 7-25) are referred to as f_2 factors.

Table 7.9 - Pier Shape Factor f_2

Shape in Plan	Ratio length/width	f_2
Circular	1	1.00
Lenticular	2 3 4 7	0.97 0.76 0.67 0.41
Parabolic nose	-	0.80
Triangular 60°	-	0.75
Triangular 90°	-	1.25
Elliptic	2 3	0.91 0.83
Ogival	4	0.86
Rectangular	2 4 6	1.11 1.40 1.11

Scour at non-cylindrical piers will vary with the direction of the oncoming flow, or angle of attack. Factors, referred to as f_3 factors, which may be used for adjusting for oblique flow, are given in Figure 7.10 (page 7-26). Hence, for non-cylindrical piers in oblique flow, the local scour may be estimated from :

$$\textit{scour depth} = d_s\, f_2\, f_3 \qquad \textbf{(7.14)}$$

where d_s = scour depth at cylindrical pier calculated from the appropriate equations selected from the equations in Table 7.7 (page 7-22)

f_2 = factor to account for pier shape

f_3 = factor to account for oblique flow

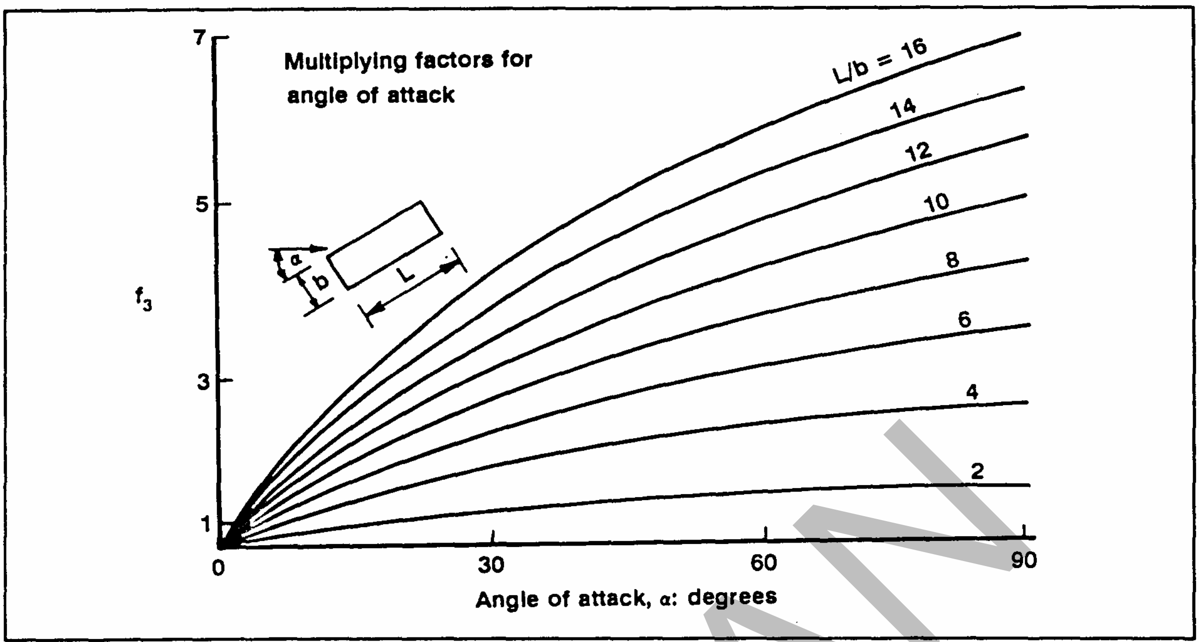


Figure 7.10 - Variation of f_3 with Angle of Attack α

iv. Pile Groups

Bridge piers are commonly founded on groups of piles. The pile cap is usually at or above the general scour level and is generally of larger plan dimensions than the pier. The flow pattern for this situation is therefore different from that for a plain cylindrical pier (see Section 7.6.3.a.i, page 7-18) :

- the downward component of flow at the pier nose will be deflected horizontally at the pile cap so tending to prevent the formation of the horseshoe vortex
- a complicated flow pattern sets up within the pile group.

No generalised guidelines are available for estimating scour around pile groups. A conservative estimate may be obtained by assuming that the group effectively becomes a single solid pier of the dimensions formed by the outermost piles in the group.

b. Local Scour at Abutments and Training Works

Abutments and training works can be subjected to a wide range of approach flow conditions of varying complexity which complicate any attempt to produce generalised guidelines for estimating scour. In any given design situation it is therefore recommended that scour depths should be estimated either from data collected at similar river works in the same locality or from model investigation.

An estimate of scour may be obtained by first estimating the *general scour* level; this may be taken as the bed level corresponding to the mean level of flow calculated as detailed in Section 7.5 (page 7-8) of this manual. To then obtain the maximum depth, the mean depth

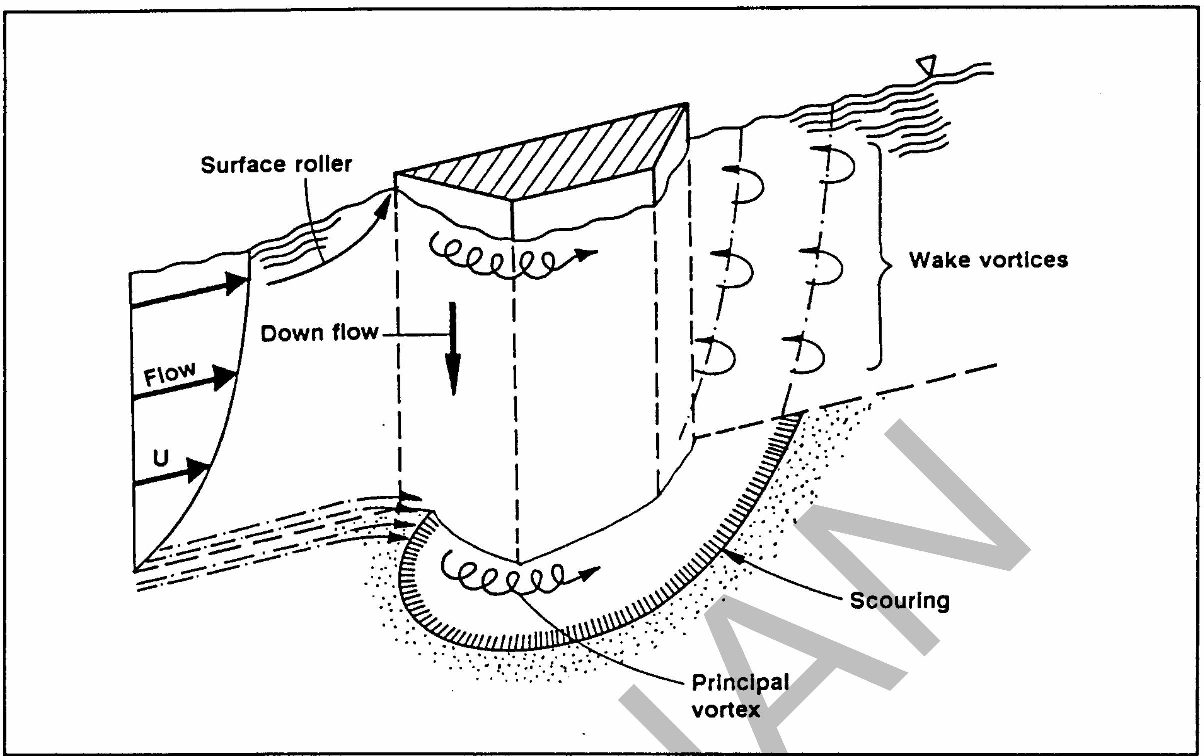


Figure 7.11 - Flow Pattern at a Typical Abutment

is factored by a multiplier selected from Table 7.10 (page 7-27). The multipliers apply to sand bed rivers but, for approximate indications, they may be applied to gravel and cohesive stream beds.

In the case of *spill-through* abutments with revetments, the scoured level may be obtained by applying the appropriate multiplier for banks to the *general scour* level for the waterway. In the case of abutments which protrude into the river flow, it is more difficult to give design guidelines. A conservative approach would be to assume the scoured level to the lower of the scour level estimated for piers and the level calculated by factoring the general scour level with a multiplier of ***2.25***, a multiplier applying to stream locations adjacent to walls.

Table 7.10 - Multipliers for Estimating Scour Depth at Abutments and Training Works

Nature of Location	Multiplier
Nose of groynes or guidebanks	2.0 - 2.75
Flow impinging at right angles on bank	2.25
Flow parallel to bank	1.5 - 2.0

c. Worked Example

Example 7.1 (page 7-28) illustrates a typical calculation for local scour depth estimation.

Example 7.1 - Local Scour Depth Estimation

Step	Local Scour Depth Estimation Procedure
Detail	Stream classification : *type* = *sand stream bed* Depth of upstream flow : y_0 = *3.00* *m* Approach velocity : U_0 = *0.87* *m/s* Median particle size : D_{50} = *0.78* *mm* Pier length : L = *8.85* *m* Pier width : b = *2.44* *m* Critical approach velocity : U_c = *0.30* *m/s* Angle of attack : α = *10* *°*
Step 1	Calculate Froude Number $F = U_0/\sqrt{(gy_0)} = 0.16$
Step 2	From Table 7.9 (page 7-25) factor to account for pier shape $f_2 = 1.11$
Step 3	From Figure 7.10 (page 7-26) factor to account for oblique flow $f_3 = 1.3$
Step 4	Calculate entrainment function $F_E = yS(s-1)^{-1}D^{-1} > 0.1$
Step 5	From Figure 7.8 (page 7-19) it is evident that sediment transporting conditions prevail
Step 6	Referring to Table 7.7 (page 7-22) for appropriate equations to estimate the depth of scour we find *scour depth* $= 1.11\ y_0^{0.5}\ b^{0.5}\ f_2\ f_3$ $= 1.11 \times 1.73 \times 1.56 \times 1.11 \times 1.3$ $= 4.3$ m The estimated local scour depth is 4.3 m for this pier

7.7 CONSTRICTION SCOUR

Constriction scour occurs when the bridge and its associated training works so constrict or realign natural stream flows that an artificial waterway opening is created, bounded on each side by road approach embankments or guide banks. The problem is then to estimate scoured depths due to the design flow passing through the controlled waterway opening.

7.7.1 Method C1 - New Zealand Railways Method

Basis of Method

The method for estimating *constriction scour* detailed in this section is based upon that given in *Code of Practice for the Design of Bridge Waterways* produced by the New Zealand Ministry of Works and Development (Reference 7.11).

Estimation Method

This method is given in Section 7.5.1 (page 7-8) which includes both *general scour* and *constriction scour*.

7.7.2 Method C2 - Method from C.R. Neill

Basis of Method

The method for estimating *constriction scour* detailed in this section is based upon *Guide to Bridge Hydraulics* edited by C.R. Neill (Reference 7.2).

Estimation Method

The following sections detail various methods for estimating maximum constriction scour at a bridge site.

a. Mean Velocity Method

This method uses the concept of cross-sectional mean velocity as a rough criterion of general scour.

Table 7.11 - Mean Velocity Method for Estimating Constriction Scour

Step	Scour Estimation Procedure
Step 1	a. Obtain representative cross-section of stream channel and stream slope. By direct measurement or by using the slope-area method as outlined in Section 6.2 determine stage-discharge relationship. b. Calculate mean velocity of flow in the main channel for design discharge.
Step 2	a. Measure the net water area of the proposed waterway opening under design discharge, before scour, and calculate the cross-sectional mean velocity through the opening. b. If this is significantly greater than the mean velocity of flow in the main channel as calculated in Step 1, constriction scour should be allowed for.
Step 3	Determine by trial the average constriction scour level, assuming a trapezoidal cross-sectional shape, that will make the mean velocity through the waterway opening equal to the estimated average velocity at design discharge, as estimated in Step 1.
Step 4	Re-distribute the trapezoidal cross-sectional area to give the worst expected cross-sectional shape and lowest elevation of constriction scour, as described in Section 7.7.2.c (page 7-34).

b. Competent Velocity Method

This method depends on the hypothesis that general scour will proceed in the waterway opening until the mean velocity is reduced to a value just capable of moving the bed material exposed at the scoured depth. This velocity is designated the competent velocity. In channels carrying substantial bed loads this principle is very conservative, but it can still be applied to estimate a maximum limit to scour.

Table 7.12 - Competent Velocity Method for Estimating Constriction Scour

Step		Scour Estimation Procedure
Step 1	a.	Calculate the mean velocity through the waterway opening at design discharge, assuming no scour.
	b.	Determine the corresponding depth of flow and median diameter (d_{50}) of bed material by weight.
Step 2	a.	For non-cohesive materials, compare the calculated mean velocity with the competent velocity indicated by Figure 7.12 (page 7-32), using the appropriate flow depth and d_{50}.
	b.	For cohesive materials, compare the calculated mean velocity with the competent velocity given in Table 7.13 (page 7-33).
	c.	If the calculated mean velocity significantly exceeds the competent velocity, constriction scour should be allowed for.
Step 3	a.	Determine by trial the average constriction scour level, assuming an appropriate cross-sectional shape, that will make the mean velocity through the waterway opening equal to the competent mean velocity for the material exposed at that level, as given by Figure 7.12 (page 7-32) or Table 7.13 (page 7-33).
	b.	The appropriate average depth of flow after scour should be used in selecting the competent velocity. For widely graded granular mixtures where some degree of paving might be expected as scour proceeds, it may be appropriate to select the competent velocity corresponding to a grain size larger than d_{50}, but the size selected should not be greater than d_{80}.
Step 4	Re-distribute the trapezoidal cross-sectional area to give the worst expected cross-sectional shape and lowest elevation of constriction scour, as described in Section 7.7.2.c (page 7-34).	

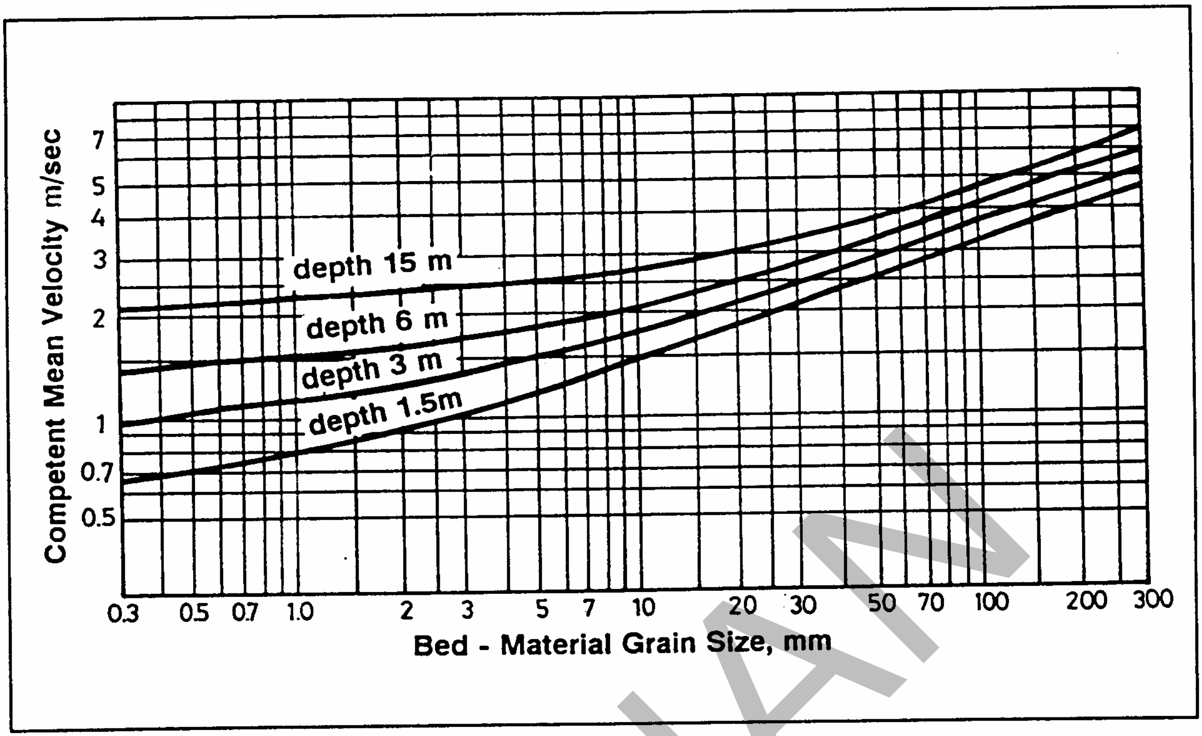


Figure 7.12 - Suggested Competent Mean Velocities for Significant Bed Movement of Non-Cohesive Materials in Terms of Grain Size and Depth of Flow

Table 7.13 - Tentative Guide to Competent Velocities for Erosion of Cohesive Materials

Depth of Flow (m)	Competent Mean Velocity (m/s)		
	Soil Erodibility		
	High *very soft to soft clays*	**Medium** *firm to stiff clays*	**Low** *stiff to hard clays*
1.0	0.5	0.9	1.6
1.5	0.6	1.0	1.8
3.0	0.65	1.2	2.0
6.0	0.7	1.3	2.3
15.0	0.8	1.5	2.6

NOTES

1. *This table is to be regarded as rough guide only in the absence of data based on local experience. Account must be taken of the expected condition of the material after exposure to weathering and saturation.*

2. *It is not considered advisable to relate the suggested low, average, and high values to soil shear strength or other conventional indices, because of the predominating effects of weathering and saturation on the erodibility of many cohesive soils.*

3. *Soil consistency can be judged from the following field test applied at or near the soil's natural water content.*

Consistency	***Field Test***
very soft	*easily penetrated several centimetres by fist*
soft	*easily penetrated several centimetres by thumb*
firm	*moderate effort required to penetrate several cms*
stiff	*readily indented but penetrated only by great effort*
very stiff	*readily indented by thumbnail*
hard	*indented with difficulty by thumbnail*

c. Redistribution of Cross-Sectional Area

Estimation of maximum constriction scour is a very approximate and uncertain process. In general the redistribution should be done graphically with reference to Figure 7.13 (page 7-35), bearing in mind the following points.

i. In non-alluvial streams with cohesive or semi-cohesive beds that are expected to scour to only a limited degree as a result of flow constriction, it is probably sufficient to redistribute the estimated net area of scour below natural bed, as illustrated in Figure 7.13a. The redistribution need not take the triangular form shown.

ii. In intermediate types of stream with limited bed transport, the area to be re-distributed may extend up to low water level or any higher level that may appear appropriate, depending on an assessment of the level to which the channel bed is likely to shoal (Figure 7.13b).

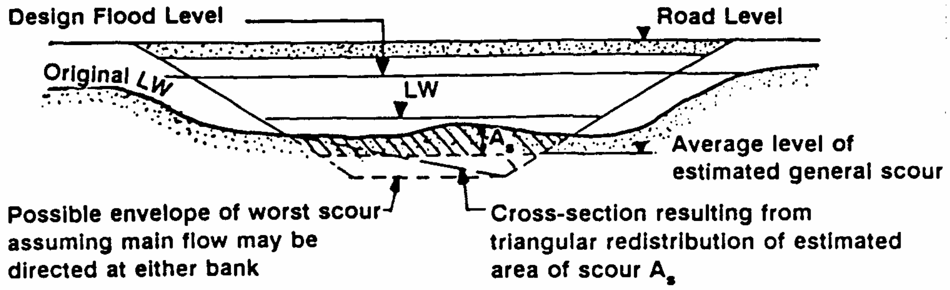
iii. For wide alluvial streams where the surface and bed widths are not greatly different, and where shoaling and other bed changes may occur over the whole depth up to flood level the average scoured depth below flood level should be multiplied by a factor of ***1.4*** or more, as illustrated in Figure 7.13c.

iv. In Figure 7.13 (page 7-35) it is assumed that side slopes of the scoured area are maintained at a ***1.5 to 1*** angle of repose. This would normally be ensured by providing rock protection where necessary (see Section 8.3).

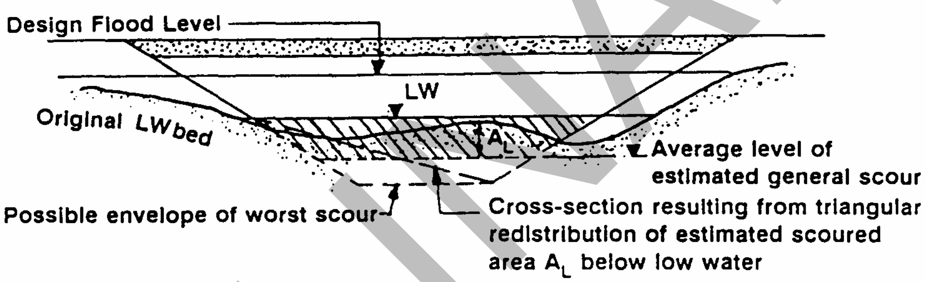
v. The cross-sectional shape will depend on the approach alignment and on the layout of training works. A section on a sharp bend will tend to adopt a more or less triangular form below the highest level of shoaling. A straight alignment with parallel guide banks will favour retention of a more trapezoidal section. Triangular or irregular sections may, however, develop in alluvial channels with straight alignments, as sand bars pass through the opening.

vi. On a bend the deepest point often tends to remain near the outer bank. In other cases it may be necessary to allow for an envelope of worst scour (Figure 7.13a and b), assuming the deepest point can shift from side to side.

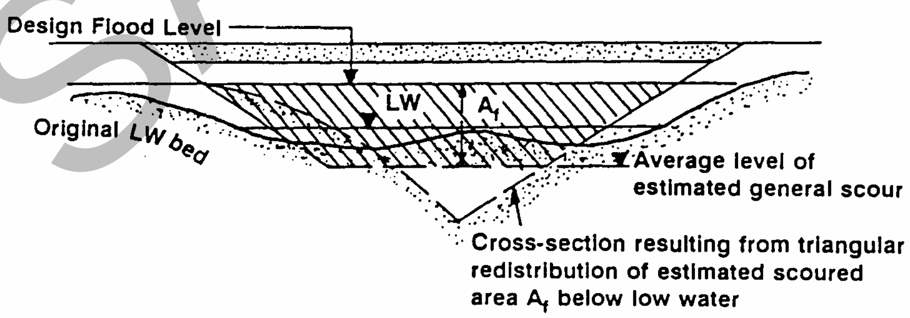
In view of a general lack of data on this question, little further guidance can be given. Considerable weight should be given to local experience where it has been accurately recorded.



a. Redistribution of Net Area of Scour Only for Non-Alluvial Streams with Erodible Beds



b. Redistribution of Scoured Area below the Low-Water Level for Semi-Alluvial Streams with Limited Bed Movement



c. Redistribution of Scoured Area below the Design Flood Level for Alluvial Streams with Highly Mobile Channels

Figure 7.13 - Various Alternatives in Graphically Redistributing the Cross-Sectional Area of a Scoured Waterway Opening

7.8 DEGRADATION SCOUR AND AGGRADATION

Definition

Degradation and *aggradation* (Reference 7.14) are the lowering and raising of the stream bed, respectively, over relatively long reaches and long time periods.

Quantification

Quantification of the degradation and aggradation component of the estimated scour depth relies on estimating the sediment supply or transport capacity of the upstream reach of the stream.

Design Consideration

Naturally occurring degradation is a long term effect and for most bridges it is unlikely to be of importance during the life of the bridge. Artificially induced degradation scour or aggradation due to an increase or decrease in stream flow, or reduction or increase in sediment supply could be a problem, however, and should be considered when designing a bridge. As there is no easy method for predicting degradation scour or aggradation of the stream bed, specialist advice should be sought where it is likely to occur.

Qualitative Determination

A qualitative determination of degradation scour or aggradation can be carried out and may be based on the following :

- **Historic Data**

  Collection and comparison of all historic data relating to the bridge site. In particular, historic stream bed profiles should be studied, if available, to detect any trend in degradation or aggradation. Less detailed information may be available from elevation of pipeline crossings and highway bridges. With knowledge of the elevation of these structures, it is relatively simple to make field measurements of present stream bed elevations. Additionally, the construction plans for these structures can provide historical information. The invert elevations at the time of construction are usually provided on the plans or can be deduced from the given information.

- **Field Inspections**

  Field inspections should be conducted upstream and downstream of the construction site. Special attention should be directed to the existence of mining operations or changes in the sediment inflow from tributaries. For example, gravel operations induce a headcut (lowering of the stream bed) which can potentially migrate upstream through the construction site. Alternatively, an upstream tributary heavily laden with sediment due to recent land use changes may cause aggradation through the construction site.

Adjustment of Scour Estimates

The results of this qualitative determination can be used to adjust the limits of protection as follows :

- **Degradation**

  If long term degradation of the stream bed is noted, the estimated scour depth resulting from the summation of the general, local and constriction scour components should be increased.

- **Aggradation**

  Should the stream be experiencing aggradation, the height of the protection may need to be reviewed and increased.

7.9 OTHER CONSIDERATIONS

7.9.1 Natural Armouring as a Limit to Scour for Gravel Stream Beds

Natural armouring may limit the scour in a gravel stream bed (Reference 7.13 and 7.14). The armouring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached through this coarse sublayer to augment the material in transport. As movement continues and degradation and scour progresses, an increasing number of non-moving particles accumulate in the sublayer. Eventually enough coarse particles accumulate to shield or *armour* the entire bed surface. When fines can no longer be leached from the underlying bed, degradation and scour is arrested.

The potential for the development of an *armour layer* can be assessed using a representative bed material composition and *Shield's criteria for incipient motion* :

$$D_c = \frac{\tau_c}{0.047\ (s_s - s_w)} \qquad \textbf{(7.15)}$$

where D_c = diameter of the sediment particles for conditions of *incipient motion* (m)

τ_c = critical boundary shear stress (Pa)

s_s = specific weight of sediment (Pa/m^3)
= density of sediment (kg/m^3) x g (m/s^2)

s_w = specific weight of water (Pa/m^3)
= density of water (kg/m^3) x g (m/s^2)

g = acceleration due to gravity (9.81 m/s^2)

To determine the size of the armouring particle for a given set of conditions, the critical shear stress is determined by :

$$\tau_c = \frac{V^2\ n^2}{y^{1/3}} \qquad \textbf{(7.16)}$$

where V = design flow velocity (m/s)

n = Manning's roughness coefficient

y = design flow depth (m)

Having determined τ_c, the armouring size particle is determined by Equation (7.15) . If no sediment of the computed size or larger is present in significant quantities in the stream bed, armouring will not occur. Armouring is likely to occur when the particle size computed from Equation (7.15) is equal to or smaller that the D_{95} of the bed material.

By determining the proportion of bed material equal to or larger than the armour particle size (D_c), the depth of scour necessary to establish and armour layer (d_a) can be calculated from :

$$d_a = y_a \left(\frac{1}{P_c} - 1 \right) \qquad \textbf{(7.17)}$$

where d_a = depth of scour necessary to establish an armour layer (m)

y_a = thickness of armour layer (m) = $2\ D_c$

P_c = decimal fraction of the material coarser than the armouring size D_c

D_c = armour particle size (m)

Should the maximum predicted scour depths exceed the armouring depth, it is likely that an armour layer will develop. It should be recognized, however, that the development of an armour layer will not occur uniformly across a stream bed but tends to begin along the *thalweg* and at other points of natural scour in the stream. Caution should be used in limiting scour protection along stream embankments, groynes or abutments under the assumption that a uniform armour layer will be created. If a uniform armour layer is not present or if one fails to develop at a predicted depth during a design flow event, the protection measure could be undermined by scouring action.

Example 7.2 (page 7-39) illustrates a typical calculation for armouring depth estimation.

Example 7.2 - Armouring Depth Estimation

Step	Armouring Depth Estimation Procedure
Detail	a. Critical particle size D_c = ***0.038 m***. b. Representative stream bed material gradation curve shows that this size of particle to be the D_{90} size.
Step 1	Calculate the depth for formation of an armour layer : $d_a = y_a\left(\frac{1}{P_c} - 1\right) = 2 \times 0.038\left(\frac{1}{0.1} - 1\right) = 0.69\ m$
Step 2	If the maximum predicted scour depth exceeds the armouring depth of 0.69 m, it is likely that the armour layer will develop.

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DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 8

SCOUR PROTECTION

FEBRUARY 1993

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8. SCOUR PROTECTION

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8. SCOUR PROTECTION

8.1 INTRODUCTION

This section of the manual gives guidelines for design to resist scour as well as methods of computing geometric parameters of typical scour protection arrangements for foundations, abutments, embankments, waterway invert and waterway training works.

8.2 PIERS

8.2.1 Spread Footings in Soil

When there is any risk of scour undermining spread footings, deep foundations in the form of piles or caissons should be used.

8.2.2 Footings on Erodible Rock

Serious problems and failures have been encountered with piers founded on erodible rock. Footings should be founded at depths sufficient to prevent undermining and to protect the interface between the structure and its foundation. No method presently appears available for prediction of the severity of the problem of rock scour other than experience with structures in the same area founded on similar material. Because scour is aggravated by increased velocities and turbulence produced by flow around the pier, any attempt to hydraulically streamline the pier base will obviously relieve potential problems. Rock protection placed around the base may also lessen scour.

8.2.3 Piling

Piling driven deep below the stream bed affords a degree of protection against scour. This feature must not be taken for granted where scour is expected to depths considerably below the natural stream bed. A structural system must be provided to resist stream flow forces under the scoured condition and to provide stability. The piles need to be of sufficient length to support the structure after the scour has occurred.

8.2.4 Rock Aprons

The designer has the choice of designing bridge foundations in streams to be adequately supported below the lowest estimated level of scour, or of designing suitable protection works, such as a rock apron, which will limit the depth of scour and permit design for support below the level of the protection works. An example of a typical foundation is shown in Figure 8.1 (page 8-1) in which a rock apron limits the depth of scour to the *general scour level*, the level at which protection is usually provided.

In the case of rock apron design a model investigation may be appropriate to determine the size of stone and the extent of the apron. Otherwise design may be based on experience of similar installations in the same locality or from theoretical considerations. Neill (Reference 8.2) recommends that the apron should be laid below the *general scour level*, that it should project around the nose of the pier by ***1.5*** times pier width, and that it should be equal in thickness to twice the D_{50} size of stone (D_{50} = median particle size of riprap stone). Gales (Reference 8.13) has recommended wedged-shaped pitching in plan around piers as shown in Figure 8.2 (page 8-2). This is a more conservative solution than that proposed by Neill but

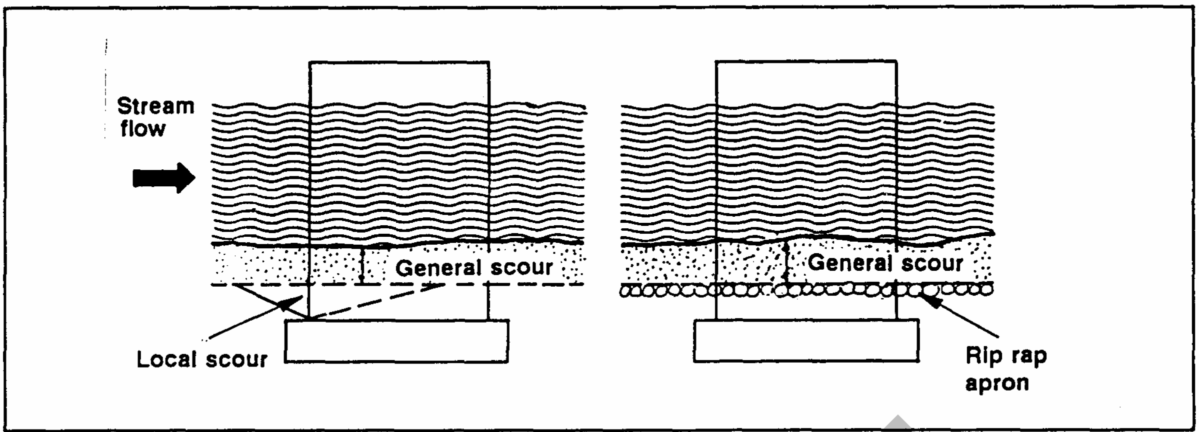


Figure 8.1 - Protection of Pier Foundations

some economy could probably be achieved by reducing the quantity of stone pitching at the tail of the pier where scour conditions are less severe than at the nose.

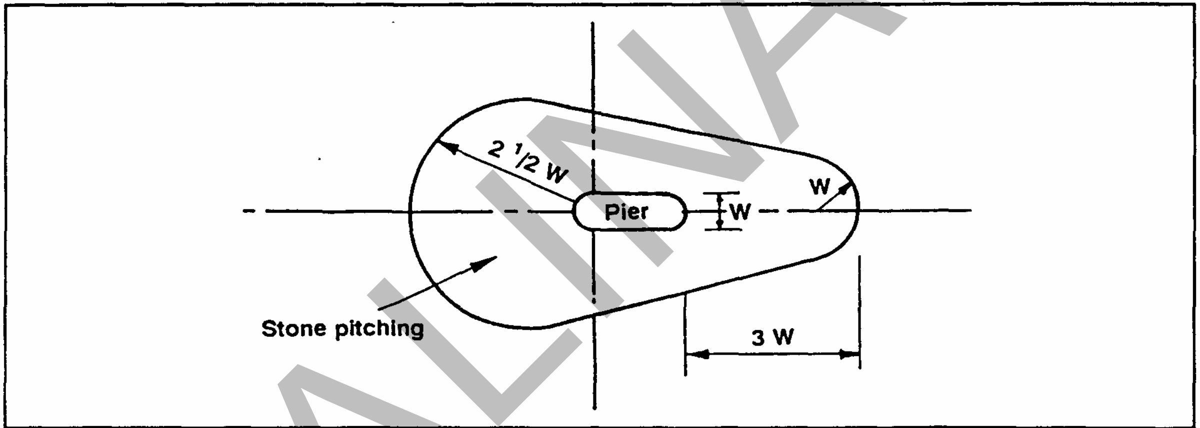


Figure 8.2 - Plan of Pier Foundation Protection

8.3 ABUTMENTS

8.3.1 Guide Banks

Where approach embankments direct considerable flood plain flow through the bridge opening, a guide bank, properly proportioned, is effective in reducing the gradient and velocity along the embankment by moving the mixing action of the converging flow away from the abutment to the upstream end of the guide bank as shown in Figure 8.3 (page 8-2), thus protecting piers and abutments from the effects of scour.

Guide banks may be designed so that the entire waterway under the bridge is utilised and the depth of scour in the vicinity of the bridge abutments and at adjacent piers is reduced.

Guide banks only redistribute areas of scour. They do not effect afflux significantly.

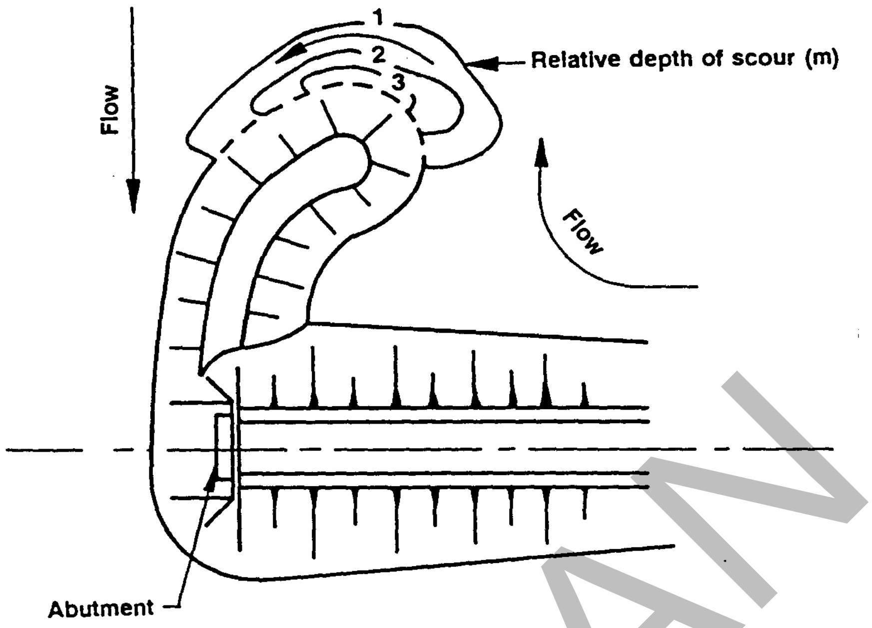


Figure 8.3 - Scour Around a Guide Bank

Three principal considerations are involved in proportioning a guide bank :

- geometry
- height
- length

a. Geometry

A guide bank in the form of a quarter of an ellipse, with ratio of major (length) to minor (offset) axes ***2.5:1*** has been found to perform as well or better than any other shape tested (see Figure 8.4, page 8-5). The equation for this shape is :

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0.4\ L_s)^2} = 1 \tag{8.1}$$

b. Height

Height is based on the anticipated high water level. The guide bank should have sufficient height and freeboard to avoid overtopping and be protected from wave action.

c. Length

There are two methods available for estimating the length of guide bank. Both methods are designed to produce uniform flow under the bridge and are detailed below.

i. Method 1

This method is recommended in the *Hydraulics of Bridge Waterways* (Reference 8.12) in which the length of guide bank L_s is determined from the discharge ratio Q_f/Q_{10}, relating the flow over the left or right flood plain to a specific portion of the flow under the bridge and the average velocity through the bridge opening. L_s is determined from Figure 8.5 (page 8-6).

Definitions of the symbols used are :

Q	=	total stream discharge (m^3/s).
Q_f	=	lateral or flood plain flow (one side) measured at Section 1 (m^3/s). Section 1 is shown in Section 6 of this manual, Figure 6.1 .
Q_{10}	=	$Q/b \times 10$ = discharge in ***10 m*** of stream adjacent to abutment (m^3/s)
b	=	length of bridge opening (m)
V_{n2}	=	Q/A_{n2} = average velocity through bridge opening (m/s)
A_{n2}	=	waterway area under bridge at *Normal Water Level* (m^2)
Q_f/Q_{10}	=	guide bank discharge ratio
L_s	=	top length of guide bank (m) measured as shown in Figure 8.4 (page 8-5)

Figure 8.5 (page 8-6) is read by entering the ordinate with the proper value of Q_f/Q_{10}, moving horizontally to the curve corresponding with the calculated value of V_{n2} and then downward to obtain from the abscissa the length of guide bank required. It is recommended that, if the length read from the abscissa is less than ***15 m***, a guide bank is not required. For chart lengths from ***15 m*** to ***30 m***, it is recommended that a guide bank no less than ***30 m*** be constructed. This length is needed to direct the curvilinear flow around the end of the guide bank, so that it will merge with the main channel flow and establish a straight course down river before reaching the bridge abutment. No additional length of guide bank is required for skewed bridges.

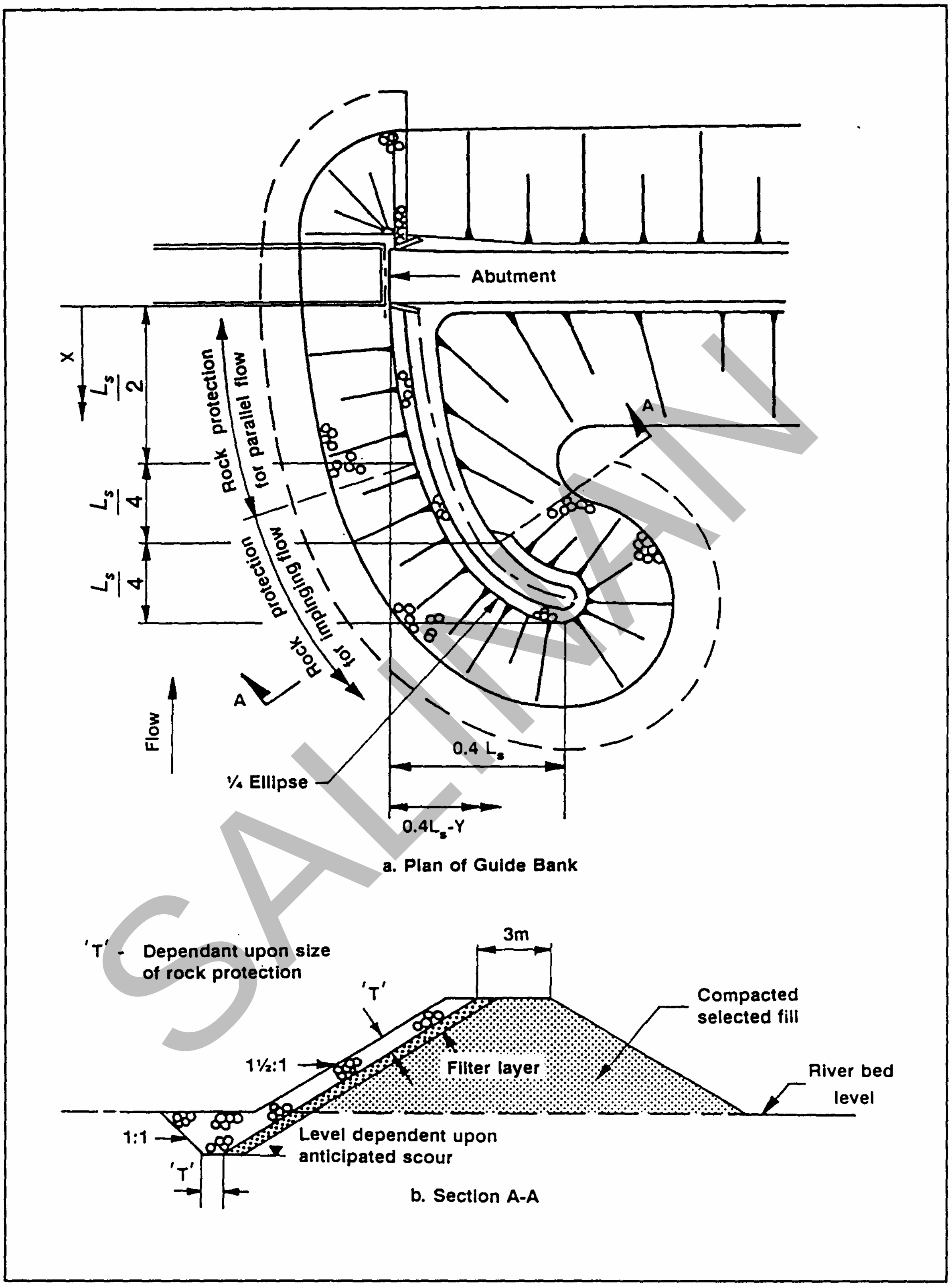


Figure 8.4 - Guide Bank Details

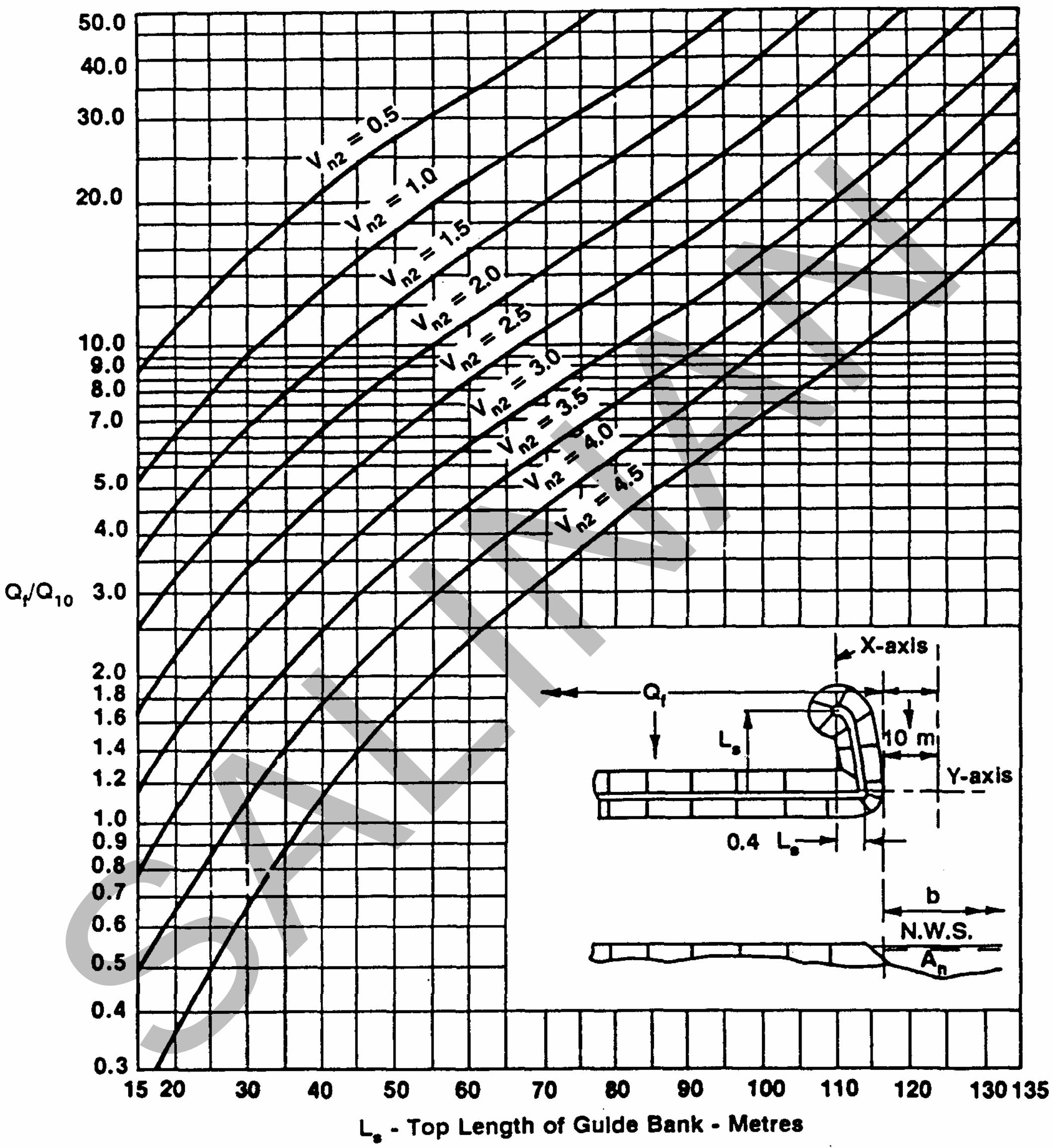


Figure 8.5 - Chart for Determining Length of Guide Banks

ii. Method 2

This method is recommended in *Guide to Bridge Hydraulics* (Reference 8.2) after Andreev (Reference 8.11) in which the procedure is as follows :

- Determine the ratio Q/Q_c, where Q is the total design discharge and Q_c is the flow in the main channel.
- For rivers with only one flood plain the upstream length of guide bank is determined using Table 8.1 (page 8-7).

Table 8.1 - Upstream Length of Guide Bank for Rivers with One Flood Plain

Q/Q_c	1.0-1.2	1.25	1.50	1.75	2.00	2.50
L_s/W	0.00	0.15	0.30	0.45	0.60	0.75

Definition of Symbols

Q	=	total design discharge	(m^3/s)
Q_c	=	flow in the main channel	(m^3/s)
L_s	=	top length of guide bank	(m)
W	=	width of main channel	(m)

- Where there is a flood plain on either side of the main channel, the length L_s, determined as shown above, is divided between the two abutments of the bridge in the proportions of the flood plain discharge ratio Q_{fl}/Q_{fr}.

  Where Q_{fl} = discharge over left flood plain (m^3/s)

  Q_{fr} = discharge over right flood plain (m^3/s)

- Adjust to suit local features.

It is also suggested by Neill (Reference 8.2) that in unstable shifting rivers guide banks should be extended downstream by approximately a third of the upstream length.

Figure 8.4 (page 8-5) shows the guide bank details including the provision of rock protection, which should be extended out from the toe of the guide bank on the river bed, so that as the scour hole forms, the rock will fall into place on the side of the scour hole to prevent undermining of the guide bank. The size of the rock protection required can be obtained from Section 8.3.2 (page 8-8).

8.3.2 Rock Protection

a. General

For embankments where scour is expected, properly designed rock riprap will afford protection against progressive erosion.

Alternatively a stone-filled wire mattress may be used (Reference 8.11) or in areas where stone is not available, sacked concrete may be used.

b. Selection of Size and Thickness of Rock

i. Method 1

The following method is based upon the Californian Division of Highways publication *Bank and Shore Protection* (Reference 8.11). The basic assumptions in determining the rock size and thickness are as follows :

The velocity ratios are

$$V_p : V_m : V_i \quad = \quad 2 : 3 : 4$$

where V_p = velocity of parallel flow along tangent bank

V_m = mean velocity of flow through bridge opening

V_i = velocity of impinging flow against curved bank

The stones are graded uniformly between specified minima for class of rock protection with two thirds heavier than minimum required on the face.

$$\text{Minimum weight of stone (kg)} \qquad W = \frac{0.011\ V^6\ SG_r}{(SG_r - 1)^3\ \sin^3(p-a)} \qquad (8.2)$$

where SG_r = specific gravity of rock

p = **70°** for randomly placed rubble

a = face slope angle from horizontal

Thickness of rock protection (m),

$$T = 0.3 \sin a \sqrt[3]{W_c} \qquad (8.3)$$

where W_c = class of rock protection (see Table 8.5, page 8-14) expressed in ***kg*** (that is W_c = ***1/4 tonne = 250 kg***)

Assuming

SG_r = ***2.65***

and $a = 1.5h : 1v = 33.7°$

then $W = 0.032\ V^6$ (8.4)

and the size and thickness of rock can be determined from Table 8.2 (page 8-9).

Table 8.2 - Design of Rock Slope Protection

Velocity (m/s)	Class of Rock Protection W_c (tonne)	Section Thickness T (m)
< 2.0	none	–
2.0 - 2.6	facing	0.50
2.6 - 2.9	light	0.75
2.9 - 3.9	¼	1.00
3.9 - 4.5	½	1.25
4.5 - 5.1	1	1.60
5.1 - 5.7	2	2.00
5.6 - 6.4	4	2.50
> 6.4	special	•

ii. Method 2

The following method is based upon *Practical Riprap Design* by Maynord (Reference 8.14). This method is relatively easy to apply and has the facility of introducing a factor of safety into the design. The basic equation for riprap design is :

$$\frac{D_{50}}{y_o} = C\ F^3 \qquad (8.4)$$

where D_{50} = median particle size of riprap stone (m)

y_o = depth upstream of bank (m)

F = Froude number = $U_o/\sqrt{(gy_o)}$

U_o = approach velocity (m/s)

C = coefficient determined from laboratory and field testing, appropriate values for which may be selected from Table 8.3 (page 8-10).

For river works, the mean channel flow velocity should be factored by the multipliers given in Table 8.4 (page 8-10) to give maximum approach flow velocity.

Table 8.3 - Coefficients for Riprap Design

Slope	Factor of Safety	Coefficient C
flat	1.0	0.22
flat	1.5	0.25
flat	2.0	0.28
3h:1v or less	1.0	0.22
3h:1v or less	1.5	0.25
3h:1v or less	2.0	0.28
2h:1v	1.0	0.26
2h:1v	1.5	0.30
2h:1v	2.0	0.32

Table 8.4 - Multipliers for Maximum Flow Velocity

Location	Multiplier
At noses of groynes and guide banks	2.0
At bends	1.5
In straight reaches	1.25

An example to illustrate the use of this method is as follows.

Data : ***y_o = 3.0 m***

U_o = 4.0 m/s

Location : straight reach

For protection around a bridge pier use a factor of safety of **2**. Thus assuming a flat bed, ***C* = *0.28*** and for straight reach the mean channel flow should be factored by ***1.25***.

$$F = \frac{U_0}{\sqrt{gy_0}} = \frac{4 \times 1.25}{\sqrt{9.81 \times 3.0}} = 0.92 \qquad \textbf{(8.5)}$$

$$\frac{D_{50}}{y_0} = CF^3 = 0.28 \times 0.92^3 = 0.22 \qquad (8.6)$$

$$D_{50} = 3 \times 0.22 = 0.66 \; m \qquad (8.7)$$

Hence stone with a median particle size of ***660 mm*** is required to protect the itself. The pier protection depends on the placement, quantity of rock, filters etc.

The grading of the riprap should follow a smooth size distribution. Simons and Senturk (Reference 8.16) recommend that the ratio of maximum size to median size D_{50} should be about ***2.0*** and that ratio between the D_{50} and D_{20} sizes should be also about ***2.0*** (D_i = size of stone such that *i%* of the stones by weight are smaller). The stone should be hard, dense and durable and be able to withstand long exposure to weathering. The thickness of the riprap should be sufficient to accommodate the largest size of stone.

A filter beneath the riprap will be essential if the underlying material is of such a grading that there is a danger of the fines being washed out through the voids in the riprap. Filters may be of gravel or of geotextiles. It has been suggested (Reference 8.16) that gravel filters of half the riprap layer thickness are adequate and that gradings should comply with the following equations :

$$\frac{D_{50}\ (filter)}{D_{50}\ (base)} < 40 \qquad (8.8)$$

$$5 < \frac{D_{15}\ (filter)}{D_{15}\ (base)} < 40 \qquad (8.9)$$

$$\frac{D_{15}\ (filter)}{D_{85}\ (base)} < 5 \qquad (8.10)$$

Guide banks and groynes require protection to prevent undermining and collapse of the slope. A common method of protecting the toe of an embankment is to use a launching apron laid horizontally on the river bed adjacent to the toe. As scour undermines the toe, the apron falls and covers the face of the scoured area. Stone sizes for the apron should be the same as for the adjacent slope revetment. Spring (Reference 8.15) recommends a thickness of ***1.25*** times the largest stone size and a horizontal length such that, in the launched position (assumed to be at a slope of ***2h:1v***), the apron extends to below the estimated scour depth. A guide bank protected with a typical launching apron is shown in Figure 8.6 (page 8-12). Alternatively, smaller stones may be used if these are encased in wire or plastic baskets to form a flexible rock mattress.

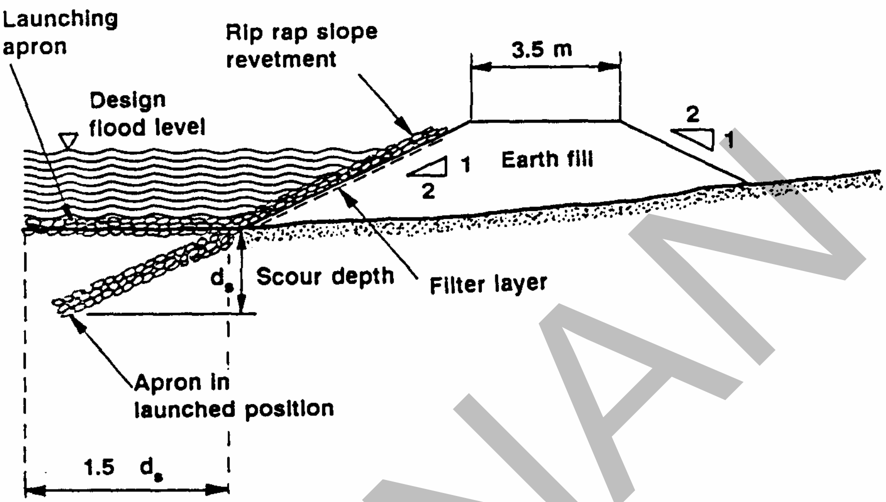


Figure 8.6 - Typical Guide Bank with Riprap Protection and Launching Apron

c. Method of Placement of Rock Protection

The thickness of the rock protection has been determined assuming the following method of placement.

A footing trench should be excavated, along the toe of the slope as shown in Figure 8.4 (page 8-5). Rock should be placed so as to provide a minimum of voids. The larger rocks should be placed in the foundation course and on the outside surface of the slope protection. The rock may be placed by dumping and may be spread in layers by bulldozers or other similar equipment.

Where filter fabrics (see Section 8.3.2.d, page 8-13) are not used best results are obtained when the embankment is raised in progressive horizontal layers. At each level the larger rocks are pushed to the face by bulldozer and where required a graded sand/gravel filter material pushed tightly in behind the rock protection, before raising the general level of the embankment to the next level.

Local surface irregularities of the slope protection should not vary from the planned slopes by more than 30 mm measured at right angles to the slope.

Example

Given that the face slope of the bridge abutment is ***1.5h:1v***, the specific gravity of the rock is approximately ***2.65*** and the mean velocity of flow through the bridge for the design flow is ***3.5 m/s***.

$$V_p = \frac{2}{3} \times 3.5 = 2.33 \ m/\text{sec}$$

$$V_i = \frac{4}{3} \times 3.5 = 4.67 \ m/\text{sec}$$

Rock protection required for parallel flow is ***facing class*** with a thickness of ***0.5 m*** and for impinging flow is ***1 tonne class*** (see Table 8.5, page 8-14) with a thickness of ***1.6 m***.

The rock protection for parallel and impinging flow should be distributed along the guide bank as shown in Figure 8.4 (page 8-5). The level to which the toe of the rock is to be carried will be dependent upon the anticipated depth of scour. The grading of the various classes or rock should be in accordance with Table 8.5 (page 8-14). A filter must be placed between the embankment fill and the rock slope protection.

d. Filter Material

A filter must be placed between the embankment fill and the rock slope protection to prevent fine embankment material from being washed out through the voids of the face stones. The filter may be a geotextile or graded sand/gravel filter.

The graded sand/gravel filter should be uniformly graded from gravel to a size (see Table 8.5, page 8-14) that will not work through the voids of the rock, or placed in two or more layers of progressively coarser sizes.

When rock slope protection consists of quarry run rock dumped into place, most of the finer material will naturally settle against the embankment face and the coarser stones will work to the outside, avoiding the need for filter material. But where the faces stones are nearly uniform in size and embankment material is vulnerable to scour, filter material will be necessary.

Embankment material should never be carried out over the rock slope protection so that the rock becomes a part of the fill. With this type of construction fill material will filter down through the voids of the large stones and the portion of fill above the rock will be lost.

Table 8.5 - Standard Classes of Rock Slope Protection

Rock Sizes	Minimum Percentage Larger Than								
	Classes							Filter Material	
	4 Tonne	2 Tonne	1 Tonne	½ Tonne	¼ Tonne	Light	Facing	No. 1	No. 2
8 tonne	0								
4 tonne	50	0							
2 tonne	—	50	0						
1 tonne	90	—	50	0					
½ tonne		90	—	50	0				
¼ tonne			90	—	50	0			
100 kg				90	—	50			
35 kg					90	—	0		
2.5 kg						90	50	0	
4.75 mm sieve							90	50	0
No. 200 sieve								95	90

8.4 WATERWAY PROTECTION AND TRAINING WORKS

8.4.1 Protection of Stream Banks

Ideally a bridge crossing should be located in a stable reach of river channel, but in many cases it will not be practicable or economical to do so. In such cases adequate measures must be taken to control the approach channel to prevent bank erosion and migrating meanders endangering the bridge construction. General descriptions of methods commonly adopted in bank protection and waterway training works are detailed below.

8.4.2 Bank and Slope Revetments

a. Revetment Types

In selecting the most appropriate type of revetment (slope protection), the degree of protection afforded, environmental acceptance, ease of installation, ease of maintenance, expected life and cost must be taken into consideration. Some of the types of revetment commonly used are stone riprap, steel sheet piling, gabions, precast concrete blocks and insitu concrete.

b. Revetment Arrangements

Lack of protection against undermining is a frequent cause of revetment failure. Figure 8.7 (page 8-17) shows the four basic methods which may be used to prevent undermining. These methods are :

- Excavate and continue the slope revetment down to an inerodible material or to below the expected scour level. This method is the most permanent, but it may be impractical or uneconomical if deep scour is expected.

- Drive a *cut-off wall* of sheet piling from the toe of the revetment down to an inerodible material or to below the expected scour level. Such walls are subject to risk of failure from earth pressure on the bank side after scour occurs on the channel side, and tend to cause deeper scour than paved slopes. The risk of failure resulting from unforeseen scour can be reduced by tying back the piling to deadmen or similar anchors.

- Lay a flexible *launching apron* horizontally on the bed at the foot of the revetment, so that when scour occurs the materials will settle and cover the side of the scour hole on a natural slope. This method is recommended for non-cohesive channel beds where deep scour is expected, as being generally the most economical.

- Pave the entire stream bed across the bridge waterway opening. This method is economical only for relatively small streams. Scour tends to occur at the downstream edge of the paving unless this is tied into a natural inerodible formation or unless an artificial stilling basin is formed. Stone sizes for riprap may be estimated as detailed in Section 8.3.2 (page 8-8). Paving may be used in cases where a launching apron is unacceptable because the scour associated with it could result in a sliding bank failure. The specified elevation of the paving must be such that the velocities through the waterway opening will be acceptable.

c. Launching Aprons

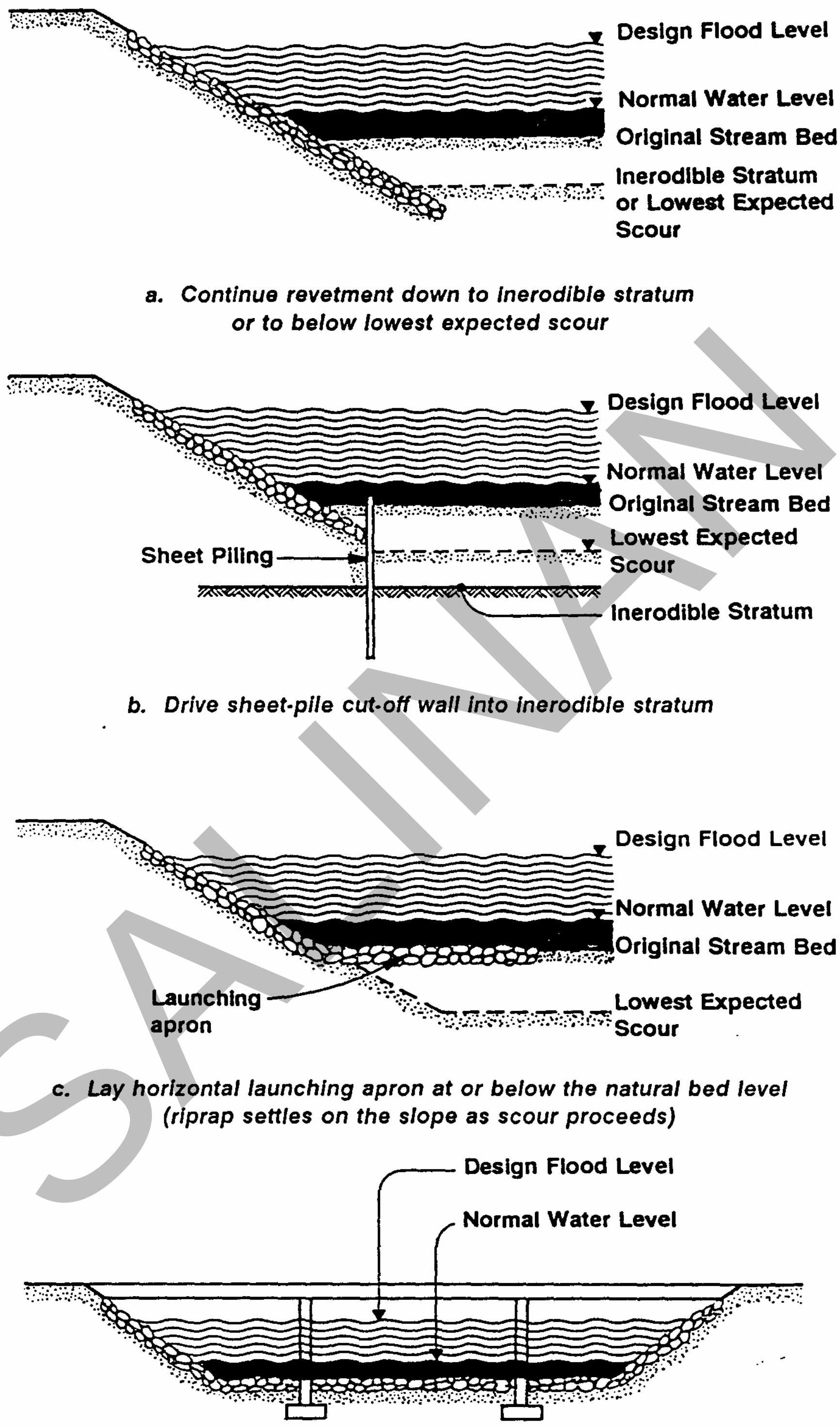
Materials used for launching aprons include stone riprap, articulated concrete matting, concrete blocks, gabions, and wire mesh mattresses filled with stone. Stone riprap is most commonly used.

In non-cohesive stream beds the design of stone aprons should be based on the stone launching to a slope of ***2h:1v***. Stone sizes should be determined as detailed in Section 8.3.2 (page 8-8). The volume of stone should be sufficient to cover the final scoured slope to a thickness of ***1¼ times*** the size of the largest stones in the specified grading (Table 8.5, page 8-14). At the nose of the guide bank or spur, there should be sufficient stone to cover the final conical surface of the scoured slope. Piers should not be located within the launching apron slope unless it is unavoidable.

Launching aprons do not perform well on cohesive channel beds where scour occurs in the form of slumps with steep slip faces. In such cases bank revetment should be continued down to the expected worst scour level, and the excavation then refilled.

d. Limits of Protection

Aprons must extend in plan around the noses of embankments beyond the limits of the expected scour under worst attack conditions. The limits of scour should, where possible, be determined on the basis of model tests or previous experience.



a. Continue revetment down to inerodible stratum or to below lowest expected scour

b. Drive sheet-pile cut-off wall into inerodible stratum

c. Lay horizontal launching apron at or below the natural bed level (riprap settles on the slope as scour proceeds)

d. Pave the entire bed of the bridge waterway opening (elevation of paving must be low enough to ensure acceptable velocities)

Figure 8.7 - Methods of Protecting Bank Revetment Against Undermining

8.4.3 Groynes

Purpose

Groynes have a number of functions in river training works but, when used in training work at bridge crossings, they will usually be required either to control the migration of a meander and channel flow through the bridge opening, or to control erosion of the river banks.

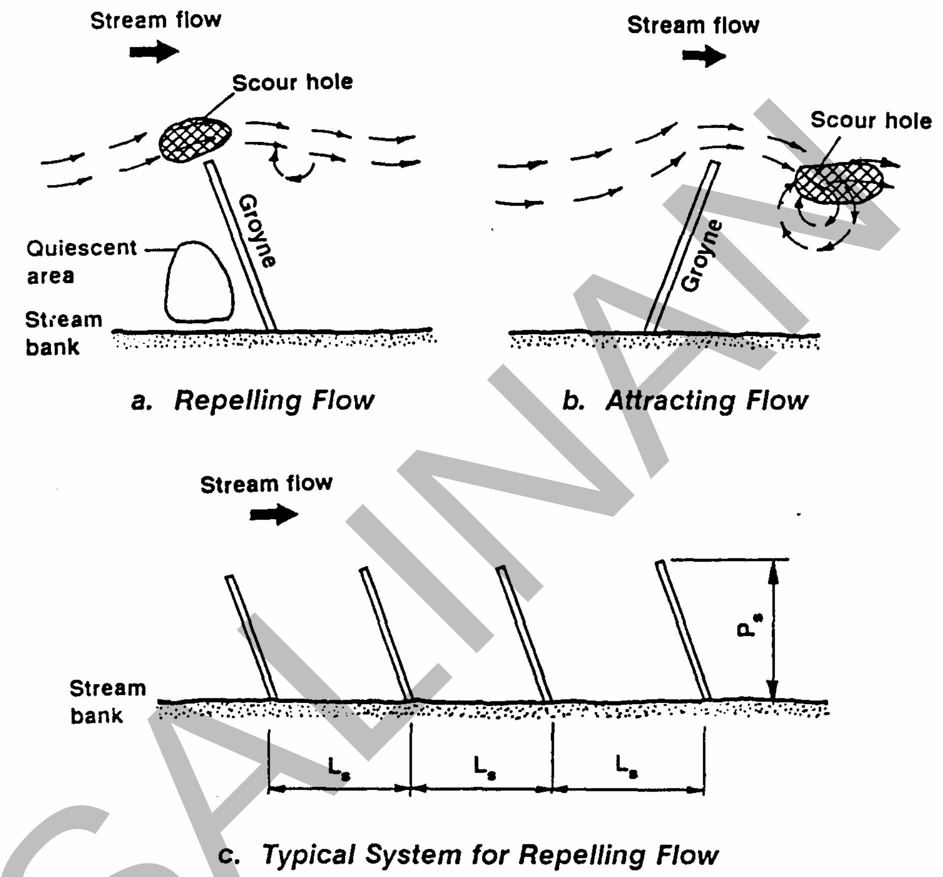


Figure 8.8 - Typical Groyne Arrangements

Location

Groynes may be concentrated upstream or downstream of the point to be protected either to repel or to attract flow (see Figure 8.8a and Figure 8.8b, page 8-18). They are used singly to repel flow and in groups to attract flow. They may also be used in groups to deflect flow, thus protecting a bank, without actually repelling the flow to the opposite bank.

Spacing

The following equation may be used as a guide in spacing groups of deflecting or attracting groynes :

$$L_s < \frac{C\, y^{1.33}}{2g\, n^2} \tag{8.12}$$

where L_s = spacing between groynes (m)

C = a constant (approximately = 0.6)

y = mean depth of flow (m)

n = Manning's roughness coefficient

g = acceleration due to gravity ($9.81 m/s^2$)

Other approximate rules for the spacing of groups along a straight river bank may be expressed as :

$$L_s = 4.0P_s \quad to \quad 4.5P_s \tag{8.13}$$

and

$$L_s = 1.0B \quad to \quad 2.0B \tag{8.14}$$

where P_s = length of groynes (m), measured normal to the river bank

B = mean channel width (m)

The spacings given by Equations (8.13) and (8.14) may be increased for banks on the inside curve of a bend and decreased on the outside of the curve.

Other Factors

Many factors, other than orientation with respect to the river flow, affect the function of a group of groynes. These include the crest height in relation to bankfull height, whether the heights of all groynes are the same relative to water surface level or whether the heights increase or decrease along the channel, and whether the crests are horizontal or inclined downwards towards the nose of the groyne. These complexities make reliable design difficult without a hydraulic model study, in all but the simplest cases.

Protection of River Banks

When groynes are used to protect a river bank from erosion, they are usually concentrated upstream and their length chosen to achieve the most economic system. Short groynes demand close spacing but the number may be decreased by increasing their length. The longer the groyne, however, the deeper and faster will be the flow at the nose and the more costly the construction. Economic considerations, therefore, feature strongly in the selection of groyne spacing and length, but generally groynes for bank protection will not exceed one quarter of the river width.

Repelling of Flow

When a groyne is required to repel flow to the other bank or when a series of such groynes is used on alternate sides of the channel to generate a stable sinuous pattern, the length is typically one third of the channel width. In the latter case, groynes should be spaced on opposite sides of the river, at distances apart equal to one half a meander length.

Construction Types

Groynes used to repel flow are of impermeable construction whilst those used for bank protection may be of permeable or impermeable construction. Permeable groynes are particularly useful in silt-laden rivers and quickly encourage sedimentation, so stabilising the bank. Permeable groynes may comprise a double row of timber piles filled with cut trees and have the advantage of being cheap. Other types of groyne construction are steel-piled walls, concrete walls or revetted embankments.

Groynes of embankment construction have side slopes varying from ***1.25h:1v*** to ***3h:1v***, depending on the construction material, and the head slopes from ***3h:1v*** to ***5h:1v***. Crest widths will vary from ***1 m*** to ***6 m*** depending on the scale and method of construction. Crest elevations can vary considerably, but for groynes designed for crests above the estimated flood level, the freeboard is usually between ***0.5 m*** and ***1 m***.

8.4.4 Dykes

Purpose

Dykes are embankments designed approximately parallel with the main river channel and have the function of protecting the area behind from flood water.

Design Requirements

The essential design requirements therefore are that they should be impermeable and high enough to prevent overtopping. If possible they should be located away from the high flow velocity areas, otherwise expensive revetment and groyne works will be required.

Typical Arrangements

Typically, dyke side slopes are between ***2h:1v*** and ***5h:1v***, crest widths are between ***2 m*** to ***5 m*** and crest elevations between ***1 m*** and ***2 m*** above the estimated flood level.

Model Studies

Hydraulic model investigations are usually appropriate in optimising the location of dykes.

Typical Case

A case in which a dyke construction is sometimes required is the case of a skewed road crossing over a flood plain. In the typical case shown in Figure 8.9 (page 8-21) the dyke construction is required to protect properties and the sections of road on the right bank where flood level is higher than in the bridge waterway by the hydraulic head required to drive the circulating flow.

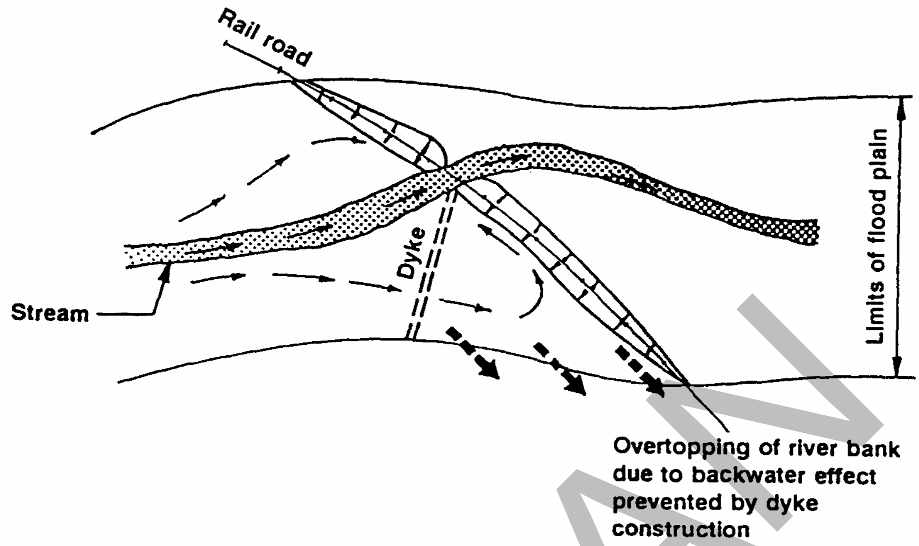


Figure 8.9 - Effect of Skewed Embankment Across Flood Plain

8.4.5 Guide banks

Purpose

Guide banks are used to protect the bridge and approaches by guiding and confining the flow through the bridge opening. Two guide banks are generally required when the bridge opening is located in the middle of a wide flood plain. However, in cases where the river meander has been confined by natural control points (that is, outcrops of inerodible material) on one side of the river, a single guide bank may be sufficient (see Figure 8.10, page 8-21).

Design Requirements

In the design of guide banks their plan shape, length, cross-section and method of construction must be considered (see Section 8.3.1, page 8-2). There is, however, no generalised approach to their design and much of the published information is in the form of general guidance only.

A variety of plan shapes may be selected for the guide banks. For example, they may be straight or curved, parallel or converging, of equal or unequal length, etc. The selection of the most suitable plan shape depends upon the site situation and relies chiefly on past experience. In addition, model studies may be necessary.

Length of Guide Bank

The upstream length of the bank should be sufficient to prevent the formation of a meander bend which will endanger the approach embankment, and be sufficient to align the flow parallel to the bridge piers (see Figure 8.10, page 8-21). The lengths may be assessed by

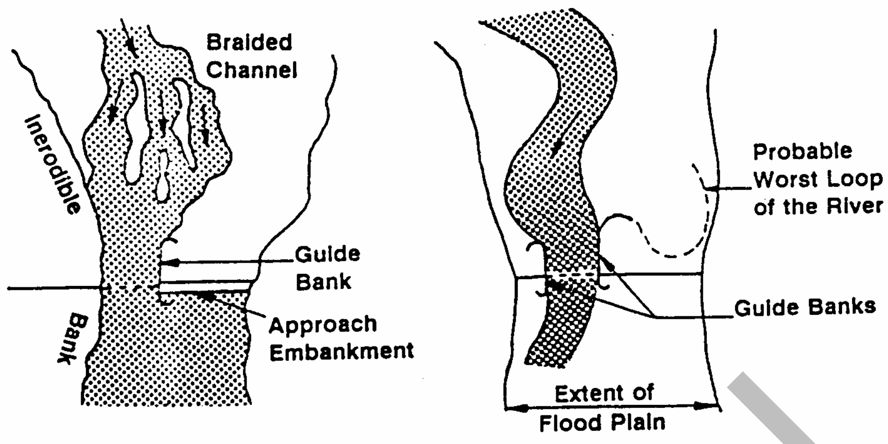


Figure 8.10 - Use of Single and Twin Guide Banks

examining the river upstream of the bridge to determine the most acute bend in the meander system and fitting it to the end of the head of the guide bank. The length of the bank can then be chosen so that meander does not endanger the approach embankment.

Extent of Guide Bank

Neill (Reference 8.2) suggests that in unstable meandering rivers the guide banks should extend upstream from the bridge by three quarters of the bridge waterway width and downstream by one quarter. The bridge waterway width is defined as the clear distance between abutments less the width or piers projected on to a plane at right angles to the direction of flow. Spring (Reference 8.15) referring to meandering alluvial rivers on the Indian sub-continent, recommends that the upstream bank should be equal to or 10% longer than the bridge waterway width and the downstream bank between one tenth and one fifth of the waterway width. Considerably shorter guide banks are recommended by Andreev (Reference 8.11 reported by Neill, Reference 8.2) for flood plain rivers with well defined channels. Andreev's recommendations are based on the ratio of the design discharge to the channel discharge according to Table 8.1 (page 8-7).

The total upstream guide bank length is proportioned between the right and left banks in the ratio of the right and left flood plain discharges. In the case where there is only one guide bank, the total length given is the upstream length of the single bank. The downstream length of the guide bank is made equal to approximately one third of the upstream length.

Heads of Guide Banks

Some guidance on the design of the heads of guide banks in sand channels is given by Spring (Reference 8.15), indicating that radii in the range ***150 m - 250 m*** with sweeps of between ***120°*** and ***145°*** are appropriate.

Typical Cross-Section

A typical cross-section for a guide bank is shown in Figure 8.4 (page 8-5) and Figure 8.6 (page 8-12). Generally the bank should extend above the estimated flood level, with ***0.5 m - 1 m*** allowance for freeboard. In determining the level of the top of the guide bank to meet this requirement, the longitudinal variation in the surface water profile should be taken into consideration. The width of the top of the bank should be sufficient to accommodate vehicles. For embankments constructed from earthfill, slope protection will be necessary and an apron will be required to prevent erosion of the toe of the embankment. The design of slope protection and aprons is detailed in Section 8.4.2 (page 8-15).

8.5 GENERAL DESIGN PROCEDURE

Table 8.6 gives a general outline of the procedure to be taken in designing and protecting a bridge structure from damage due to scour.

Table 8.6 - General Design Procedure for Bridge Scour Protection

Step	Design Procedure
Step 1	Carry out hydrologic investigation utilising Section 5, *Hydrology*, and obtain flood frequency curve.
Step 2	Carry out hydraulic investigation and obtain stage-discharge relationship utilising Section 6, *Hydraulics*.
Step 3	Determine required bridge waterway utilising Section 6, *Hydraulics*, for flow with design recurrence interval as defined in Section 5, *Hydrology*. Determine stage-backwater and stage-velocity (through bridge opening) curves.
Step 4	Determine flow to be used to estimate depth of scour (see Section 7, *Scour Prediction*).
Step 5	Determine flood flow patterns.
Step 6	Investigate geology of the bridge site for evidence of previous scour and assess potential for scour.
Step 7	Review the types and alignment of piers and the need for guide banks, channel changes, bank or rock protection.
Step 8	Estimate the scour depths for combined flood or constriction scour and local scour for the proposed bridge piers and abutments (see Section 7, *Scour Prediction*).
Step 9	Review economics of overall design (that is, alternative size and height of bridge, embankment height and alternative protection works) against risk of damage and cost of repair or replacement and finalise design.

8.6 REFERENCES

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BRIDGE INVESTIGATION MANUAL

SECTION 9

METHODS OF SOIL EXPLORATION

FEBRUARY 1993

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9. METHODS OF SOIL EXPLORATION

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9. METHODS OF SOIL EXPLORATION

9.1 INTRODUCTION

This section of the manual outlines the methods of soil exploration, lists the soil parameters required for design along with the appropriate testing method for obtaining each parameter and details a report format for presenting the results of the soil investigation.

9.2 EXPLORATION PROGRAM

Purpose

The purpose of the exploration program is to determine the stratification and engineering properties of the soils underlying the bridge site. The main soil properties of interest are strength, deformation and hydraulic characteristics. If the soil is highly erratic, there should only be sufficient borings to establish a general picture of the foundation soil conditions. An extensive boring (and laboratory testing) program is not justified in erratic soils, and the final design should be conservatively based on the properties of the weaker soils.

Planning

In planning the exploration program the foundation engineer must have a good knowledge of current and accepted methods of both field exploration and laboratory testing and their limitations. Furthermore, full advantage should be taken of any existing information, including the foundation engineer's own previously obtained information for the area around the bridge site.

9.3 INVESTIGATION METHODS

Procedures and Tests

Recommended site investigation procedures have been given in Section 2 and an outline of the principal methods of field and laboratory soil testing are given in Sections 10 and 11 of this manual.

Table 9.1 lists these procedures and gives guidance in the selection of tests required for three broad categories of bridges (which have been defined as minor bridges, standard bridges and major bridges) with categories for foundation soil type listed as rock, non-cohesive soil and cohesive soil. Where the foundation soil type is listed as rock it is assumed that rock is close to the ground surface and is overlain by shallow depths of cohesive or non-cohesive soil.

Investigation Guidelines

Table 9.1 presents soil investigation guidelines only and considerable engineering judgement is still required to select the correct investigation and testing procedures for each bridge structure.

The choice of methods depends on :

- the importance of the structure, and
- the availability of testing equipment and personnel.

Temporary Bridges

For temporary bridges only basic site investigation and testing methods have been recommended to assess properties of importance of the seismic behaviour of the site (for example, grading test and measurement of water table level to indicate the liquefaction potential of a sand deposit).

Permanent Bridges

For permanent bridges the site investigation procedures and testing methods which are recommended are those which would be regarded as essential to obtaining a reasonable knowledge of the seismic behaviour of the site.

Important Bridges

For large and important bridges additional testing requiring more specialised equipment and skilled personnel is recommended.

Table 9.1 - Methods of Soil Investigation

FOUNDATION MATERIAL	ROCK			NON-COHESIVE			COHESIVE SOIL		
CATEGORY OF BRIDGE	1	2	3	1	2	3	1	2	3
RECONNAISSANCE									
Visual inspection	✓	✓	✓	✓	✓	✓	✓	✓	✓
Aerial photographs	✓	✓	✓	✓	✓	✓	✓	✓	✓
Past construction	✓	✓	✓	✓	✓	✓	✓	✓	✓
EXPLORATION									
Seismic survey	░	░	✓	░	░	✓	░	░	✓
Electrical resisitivity survey	░	░	░	░	░	░	░	✓	✓
Pits or hand auger bores - samples and testing	✓	✓	✓	✓	✓	✓	✓	✓	✓
Borings - samples and testing	░	✓	✓	✓	✓	✓	░	✓	✓
FIELD TESTS									
Penetration test	░	✓	✓	✓	✓	✓	░	✓	✓
Vane test	▪	▪	▪	▪	▪	▪	✓	✓	✓
Water table	░	✓	✓	✓	✓	✓	░	✓	✓
Load test	░	░	✓	░	░	✓	░	░	✓
Unconfined compression test	░	✓	✓	▪	▪	▪	✓	✓	✓
Density tests (in test pits and in fills)	▪	▪	▪	░	✓	✓	░	✓	✓
LABORATORY TESTS									
Shear box test (shear modulus) or Torsion test (shear modulus)	░	░	✓	░	░	✓	░	░	✓
Triaxial tests (Elastic modulus and strength parameters)	░	✓	✓	░	✓	✓	░	✓	✓
Grading tests	▪	▪	▪	✓	✓	✓	▪	▪	▪
Moisture content	✓	✓	✓	✓	✓	✓	✓	✓	✓
Dry density	░	✓	✓	░	✓	✓	░	✓	✓
Liquid limit	▪	▪	▪	▪	▪	▪	░	✓	✓
Plastic limit	▪	▪	▪	▪	▪	▪	░	✓	✓
Specific gravity	░	░	✓	░	✓	✓	░	✓	✓
Consolidation	▪	▪	▪	▪	▪	▪	░	✓	✓
Compaction tests (for fill control)	▪	▪	▪	░	✓	✓	░	✓	✓

LEGEND		CATEGORY OF BRIDGE	DESCRIPTION
✓	procedure or test required	1	Temporary bridges
░	not necessary	*2*	***Permanent bridges***
▪	not applicable	3	Important bridges

9.4 TESTING METHODS FOR SOIL PARAMETERS

As a guide to the selection of soil testing required on a particular site, Table 9.2 to Table 9.5 show the main soil parameters required for the following design considerations :

- bearing capacity
- settlement caused by consolidation
- slope stability
- liquefaction potential
- slumping potential
- laterally loaded piles

and list the soil testing methods commonly used to obtain these parameters.

The following general notes apply to Table 9.2 to Table 9.5 of this manual :

- Where there is more than one test listed for soil parameters, the tests are listed in decreasing order of accuracy. The choice of the type of test depends on the availability of equipment and personnel and their relative local cost.
- Where non-standard equipment is used the designer cannot use already established correlation data to obtain parameters (for example, ϕ from standard penetration tests if the sampling spoon does not have the correct dimensions) and hence he must rely on local experience.
- The *shear box test* is also known as the *direct shear test.*

Table 9.2 - Testing Methods for Slope Stability Parameters

SOIL PARAMETERS	**TESTING METHODS**		
	WEATHERED ROCK	**NON-COHESIVE SOIL**	**COHESIVE SOIL**
Shear strength *Cohesion (**c** or **c'**)* *AND* *Angle of internal friction (ϕ or ϕ')*	Triaxial ($\bar{C}\bar{U}$)	Triaxial ($\bar{C}\bar{U}$)	Triaxial ($\bar{C}\bar{U}$)
	Shear box	Shear box	Shear box
	Standard penetration and correlation	Standard penetration and correlation	Shear vane *for* c_u
			Unconfined compression *for* c_u
			Dutch cone penetrometer *for* c_u
Density / unit weight	Density	Density	Density
Moisture content	Moisture content	Moisture content	Moisture content
Water table	Water table	Water table	Water table
Porewater pressure	Piezometer	Piezometer	Piezometer

NOTES

In contrast to the laboratory tests (such as triaxial, and shear box tests) the field tests (standard penetration or Dutch cone penetrometer tests) do not give direct measurements of the soil parameters required. These parameters are obtained using established correlations available in most soil mechanics texts and are briefly summarised in Section 12 of this manual.

Table 9.3 - Testing Methods for Liquefaction Potential Parameters

SOIL PARAMETERS	TESTING METHODS
	NON-COHESIVE SOIL
Medium particle size (D_{50})	Grading
Relative density	Standard penetration test
	Field density vs minimum and maximum density
Water table	Water table
Overburden pressure	Excavate test pits with in-situ density tests
	Boring and density test on tube samples

NOTES

Liquefaction potential is only applicable to non-cohesive soils.

Table 9.4 - Testing Methods for Slumping Potential Parameters

SOIL PARAMETERS	**TESTING METHODS**	
	NON-COHESIVE SOIL	**COHESIVE SOIL**
Shear strength *Cohesion (**c** or **c**′)* *AND* *Angle of internal friction (ϕ or ϕ')*	Triaxial ($\bar{C}\bar{U}$)	Triaxial ($\bar{C}\bar{U}$)
	Shear box	Shear box
		Shear vane *for* c_u
Density	Density	Density
Moisture content	Moisture content	Moisture content
Relative density	Standard penetration	*Not applicable*
	Min. and max. density	
Sensitivity *(**c** remoulded)*	*Not applicable*	Remoulded shear box
		Remoulded shear vane
Liquid limit	*Not applicable*	Liquid limit
Plastic limit	*Not applicable*	Liquid limit

NOTES

The tests for sensitivity require the measurement of the strength of the soil (normally cohesion) after it has been worked or remoulded.

Table 9.5 - Testing Methods for Laterally Loaded Pile Parameters

SOIL PARAMETERS	**TESTING METHODS**		
	WEATHERED ROCK	**NON-COHESIVE SOIL**	**COHESIVE SOIL**
Shear strength *Cohesion (c or c')* *AND* *Angle of internal friction (ϕ or ϕ')*	Triaxial ($\overline{CU}$)	Triaxial ($\overline{CU}$)	Triaxial ($\overline{CU}$)
	Shear box	Shear box	Shear box
	Standard penetration	Standard penetration	Shear vane *for* c_u
			Dutch cone penetrometer
Horizontal subgrade modulus	Lateral load test on piles	Lateral load test on piles	Lateral load test on piles
	Load test (pressuremeter)	Load test (pressuremeter)	Load test (pressuremeter)

9.5 EXPLORATION METHODS

9.5.1 Geophysical Methods

The determination of subsurface material structure and properties through the use of borings and test pits can be time consuming and expensive. Considerable interpolation between checked locations is normally required to arrive at an areawide indication of conditions. Geophysical methods involve the technique of determining underground materials by measuring some physical property of the material and, through correlations, using the obtained values for identification. Most geophysical methods determine conditions over a sizable distance. Frequently this is an advantage over the *point* checking accomplished by borings and test pits. Most geophysical measurements can be rapidly obtained. Thus, the methods lend themselves well to the checking of large areas.

The principal geophysical subsurface exploration methods in use are seismic exploration and electrical resistivity exploration.

a. Seismic Exploration

Versatility

A seismic survey is a fast, reliable means of establishing rock profiles or the location of dense strata underlying softer materials. Seismic survey techniques using simple equipment can be used as part of a geological reconnaissance. Using more sophisticated equipment, a seismic survey can be used to help fill in the gaps in the bedrock profile between boreholes in a site investigation for a major structure and a seismic survey can thus be most useful in providing a more continuous foundation profile across the site.

Interpretation

Careful interpretation of the results of a seismic survey is required and these results should be confirmed by a test boring program. Test boring is necessary because of the approximate nature of seismic surveys (for example, depths are determined to ± 20% accuracy) and because the nature of the subsoil can only be inferred from the velocity recording. Thin layers of soft soil may be totally missed as may thicker layers of softer soil which underlie much firmer soil.

i. Seismic Reflection

The seismic reflection method proceeds by inducing impact or shock waves into the soil by exploding small charges in the soil or striking a plate placed on the soil with a hammer. The shock waves are picked up through listening devices called *geophones*. The velocity of the wave in the surface soil can then be determined by recording the time lapse of the wave travelling to the geophone. As the shock point is moved away from the geophone, some of the waves pass from the surface strata into the underlying layer and then back into the surface strata and into the geophone, enabling the computation of wave velocity in the underlying material. This method is not generally used for shallow depth exploration.

ii. Seismic Refraction

Method

The seismic refraction method proceeds by inducing an impulse in the ground and the time it takes echoes to reach a transducer is measured for varying transducer-to-impact distances. The time is plotted as a function of the distance, and if a well-defined layer exists beneath the ground's surface, there will be a characteristic break in the response curve from which the depth of the layer can be determined. This method is good for determining subsurface stratigraphy but does need a relatively sharp boundary between the layers being explored. Top-of-rock exploration, where a definite interface between the rock surface and over-burden exists, is a typical application of this method.

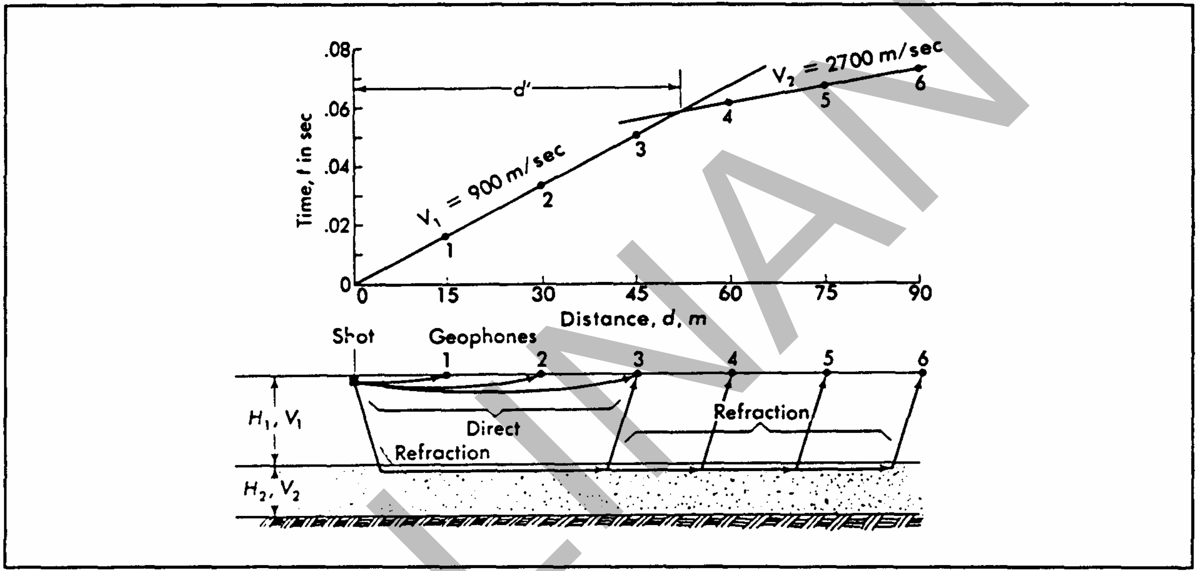


Figure 9.1 - Seismic Refraction Exploration

Limitations

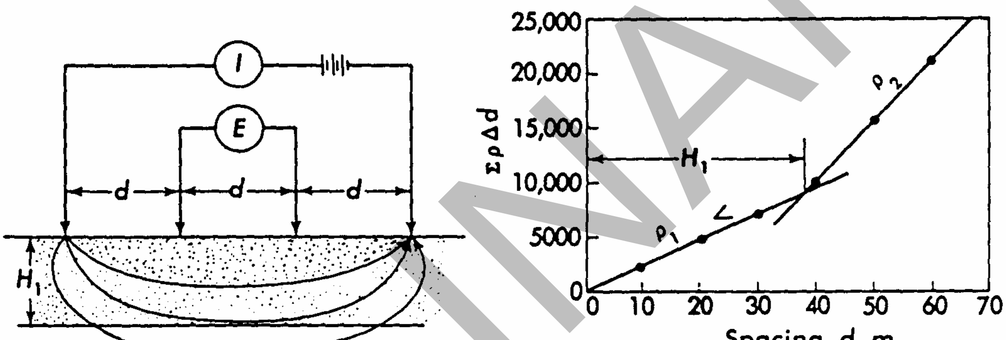
There are significant limitations to the use of the seismic refraction method for determining subsurface conditions :

- the method should not be used where a hard layer overlies a softer layer, because there will be no measurable refraction from a deeper soft layer. Refraction seismic data from such an area would tend to give a single-slope line on the travel-time graph, indicating a deeper layer of uniform material.

- the method should not be used on an area covered by concrete or asphalt pavement, because these materials will represent a condition of a hard over a soft straturn.

b. Electrical Resistivity Exploration

This method applies electrical current to the soil through electrodes and relies on the fact that any subsurface variation in conductivity alters the form of the current flow within the soil. This means that the distribution of electrical at the surface is affected and the degree to which it is affected depends on the size, shape, location, and electrical resistivity of the subsurface mass affecting it. It is therefore possible to obtain information about the subsurface distribution of various bodies from measurements of electrical potential made at the surface. Four electrodes are used, two in the outer set and two in the inner set, and they are deployed to s straight line at the soil surface. The outer set transmits the electrical current, the inner set receives it. As the four electrodes are moved along the soil surface, resistivity readings are obtained, plotted, and interpreted on the basis of known responses. This interpretation is straight forward for simple topography, but can be difficult for complex subsurface conditions.



a. Resistivity test layout, four electrode system

b. Empirical interpretation of boundary between strata

Figure 9.2 - Electrical Resistivity Exploration

9.5.2 Test Pits

Test pits are open excavations generally to 2 to 3 metres deep which allow a visual observation of the immediate subsurface layers. They also afford the opportunity of careful undisturbed sampling for later laboratory testing and for in-situ soil strength testing.

9.5.3 Boreholes

Drilling Methods

Methods of drilling include hand auger (limited depth only), machine auger, cable tool, washboring, push tube (for soft materials only), and rotary core drilling. Except for washboring, the material through which the bore is being advanced can be retrieved for visual observation. In the case of washboring, only the washings are available for inspection.

Specialised soil sampling equipment is required to obtain suitable undisturbed samples for laboratory testing.

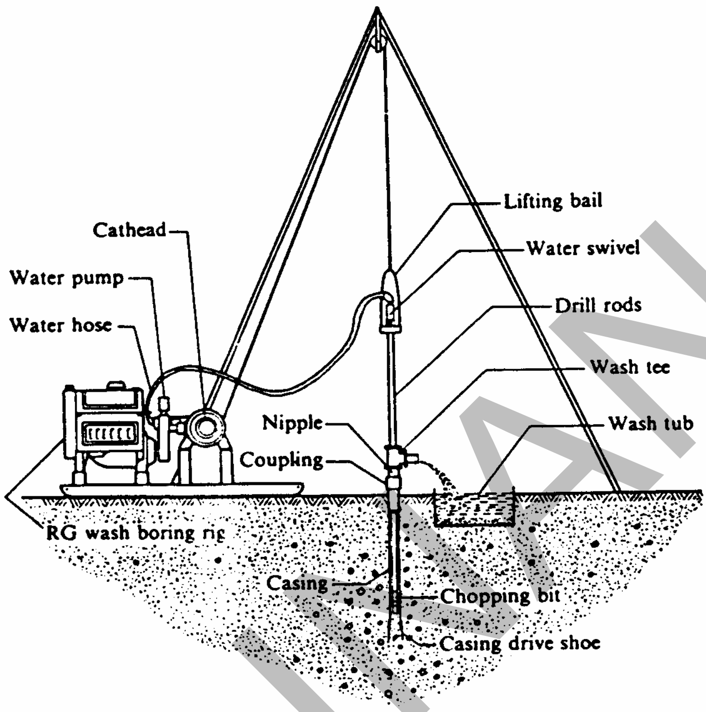


Figure 9.3 - Washboring Drilling Method

Borehole Depths

The depth to which boreholes are taken is limited by the capacity of the machinery and is determined by the requirements of the type of foundation proposed for the structure. For shallow foundations (mat and raft type) soil exploration should generally be carried out to a minimum depth of about two times the width of the footing. For pile foundations, soil exploration is required down to a layer of firm material on which the pile can be founded and the firm material should be proven for a minimum depth of five pile diameters below the proposed founding level.

9.6 SOIL INVESTIGATION REPORT

9.6.1 General

Report Format

The results of a soil investigation should be compiled in a report normally consisting of three parts :

Soil Investigation Report

- *Part 1*

  Presentation of soil investigation exploration and testing data

- *Part 2*

  Evaluation of soil investigation exploration and testing data

- *Part 3*

  Conclusions and recommendations.

Use of Report

The information contained in the *Soil Investigation Report* will normally be used for design, tendering and construction purposes.

Packaging of Report

The packaging of the report is determined by the type of contractual arrangements to be made for design and construction of the bridge works as follows :

- where the design and construction is carried out by the one party then Parts 1, 2 and 3 can be presented in one volume

- where the design is carried out by one party and the construction by another party then Part 1 should be presented in one volume available to all parties (both designer and contractor) and Parts 2 and 3 in another volume available to the designer only.

Investigation Assumptions

For all bridge categories the report should include a written statement of the assumed soil conditions and parameters. For minor bridges this statement may be very brief, for standard and major bridges a more comprehensive statement will be necessary.

9.6.2 Format of Part 1

The presentation of soil investigation exploration and testing data is a factual report which should include, but not be limited to, the following :

- purpose and scope of the soil investigation
- brief description of the bridge works for which the soil investigation report is being compiled giving information about the location of the works, its size and layout, anticipated loads, structural elements, materials of construction, etc. and also giving a statement of the anticipated bridge category for the works (minor, standard for major bridge)
- dates between which field and laboratory work were performed
- detailed description of methods used for the field and laboratory work with reference to accepted standards followed
- types of field equipment used
- names of specialised field personnel responsible for the continuous follow-up of the field work, the visual description of the samples and their handling for storage and transportation to the testing laboratory
- field reconnaissance of the general area of the project with particular emphasis of the following points :
    - history of the bridge site and its geology
    - surface observations that may be related to the bridge works from aerial photography if available
    - local experience from the area including information on groundwater, behaviour of neighbouring structures, faults, sliding areas, difficulties during excavations, etc.
    - information about quarries and borrow areas
    - seismicity of the area
- tabulation of quantities of field and laboratory work carried out
- presentation of field observations which were made by the supervising field personnel during the execution of the subsurface explorations
- data on the fluctuations of ground water table with time in the boreholes during the performance of the field work and in piezometers after completion of the field work
- compilation of boring logs with descriptions of subsurface formulations based on field descriptions and on the results of laboratory testing

- grouping and presentation of field and laboratory testing results in appendixes.

9.6.3 Format of Part 2

The evaluation of soil investigation exploration and testing data should include, but not be limited to, the following :

- review of the field and laboratory work by the soil investigation engineer. In cases where there is limited or partial data, the soil investigation engineer should state it. If, in the soil investigation engineer's opinion, the data is defective, irrelevant, insufficient or inaccurate, he can and should point this out and qualify his comments accordingly. Any particular adverse test results should be considered carefully in order to determine whether they are misleading or represent a real phenomenon that must be accounted for in the design

- tabulation and graphical presentation of the results of the field and laboratory work in relation to the requirements of the bridge works and, if deemed necessary, histograms illustrating the range of variation and distribution of the most relevant data

- determination of the depth to the ground water table and its seasonal fluctuations

- subsurface profiles showing the disposition of the various subsurface formations. Detailed description of all subsurface formations in relation to their physical properties and their compressibility and strength characteristics. Comments on irregularities such as pockets, cavities, etc.

- grouping and presentation of ranges of variation of the soil investigation data for each subsurface formation. This presentation should be in a comprehensible form which would enable the design engineer to select the most appropriate characteristic values for the design

- submission of proposals for further field and laboratory work, if deemed necessary, with comments justifying the need of this extra work. This proposal should be accompanied by a detailed program for the types of extra investigations to be carried out with specific reference to the points which have to be answered.

9.6.4 Format of Part 3

The conclusions and recommendations of the soil investigation report should include, but not be limited to, the following :

- classification of the bridge works according to bridge category (temporary, permanent or important bridge)

- differentiation between subsurface formations and selection of suitable characteristic values for the relevant computations depending on the requirements of the bridge works

- basic settlement and stability computations which will assist the design engineer

- recommendations for the most feasible and economic construction systems and techniques

- recommendations concerning problems that may be encountered during excavations, pumping operations, construction of retaining structures and ground anchors, placement of earth materials, etc.

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DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 10

FIELD TESTING

BRIDGE MANAGEMENT SYSTEM

BMS

A PROJECT JOINTLY FUNDED BY INDONESIA AND AUSTRALIA

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10. FIELD TESTING

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10. FIELD TESTING

10.1 INTRODUCTION

Field tests should be carried out during subsoil exploration to obtain a quantitative assessment of the soil encountered; such information can add greatly to the value of the investigation borehole at minor additional cost. Without such information the designer must either rely on a visual description to assess the strength of the various soil layers (with a consequently much larger margin for error in the design assumptions) or alternatively he must carry out a very extensive program of laboratory testing on undisturbed samples, but even so, the in-situ test results provide a valuable correlation with laboratory test results, particularly in the case of sensitive soils which are likely to be affected by sample disturbance.

The principal types of field testing available to determine soil parameters are :

- penetration tests
  - static penetration test
  - dynamic penetration test
- vane tests
- measurement of ground water table
- field load tests
- pressuremeter tests
- field unconfined compression tests
- in-situ soil density tests.

10.2 PENETRATION TESTS

10.2.1 General

Testing Method

Changes in ground conditions can be identified by differences in the resistance of the strata to being pierced by a *penetrometer*. Most penetrometers consist of a conical point attached to a drive rod of smaller diameter. Penetration of the cone forces the soil aside, creating a complex shear failure, resembling the point penetration of a foundation pile. The test, therefore, is an indirect measure of the in-situ shear strength of the soil.

Borehole and Non-Borehole Tests

Penetration tests can be categorised as *borehole tests* and *non-borehole tests*. In the borehole tests penetration testing is carried out at the bottom of a prebored hole. In the non-borehole tests no preboring is involved and penetration testing is carried out by driving the testing device directly into the ground with the device forming its own hole on the way down.

Static and Dynamic Tests

Two forms of penetration are used : *static* and *dynamic*. In the *static* test the point is forced ahead at a controlled rate and the force required for movement is measured. In the *dynamic* test the penetrometer is driven a specified distance by hammer blows of equal energy. The number of blows or total energy required for the specified distance is the measure of resistance. The static test is very sensitive to small differences in soil consistency. The test operation probably does not seriously change the structure of loose sands or sensitive clays.

The dynamic test is adapted to a much wider range of soil consistencies and can penetrate gravels and soft rock that would stop a static device.

Penetration Test Types

Table 10.1 lists the common penetration test types and identifies the basic test characteristics.

Table 10.1 - Penetration Test Types

Category	Form	Penetration Test Type
borehole	static	Dutch Cone Penetration Test
	dynamic	Standard Penetration Test (SPT)
non-borehole	dynamic	Dynamic Cone Penetration Test

10.2.2 Dutch Cone Penetration Test

Static Testing

The *Dutch Cone Penetration Test* is the most widely used for borehole static tests.

Testing Method

The test cone has a 60° point angle, a diameter of 35.7 mm and a projected area of 1000 mm^2. The cone is forced downwards at a steady rate (15 to 20 mm/s) through soil by means of a load from a hydraulic cylinder transmitted to solid 15 mm diameter rods. These solid rods are centrally placed within 36 mm diameter outer rods. The load acting at the top of the inner rods can be determined from pressure gauge readings and the cone resistance is taken to be this load divided by the end area.

One form of this penetrometer has an independent sleeve attached behind the cone. The force developed by friction between the sleeve and the soil can be measured independently of the cone resistance. The ratio of sleeve resistance to cone resistance is higher in cohesive soils than in cohesionless soils. This ratio helps to estimate the type of soil.

The mechanical systems for measuring the resistances vary with the manufacturer. They range from simple rack and pinion drives with spring balance weighing devices to automatic hydraulic driven machines with continuous load indicators and recorders. All are limited in the penetration force that can be developed : from half a tonne in simple equipment to several tons for large machines that are anchored to the ground.

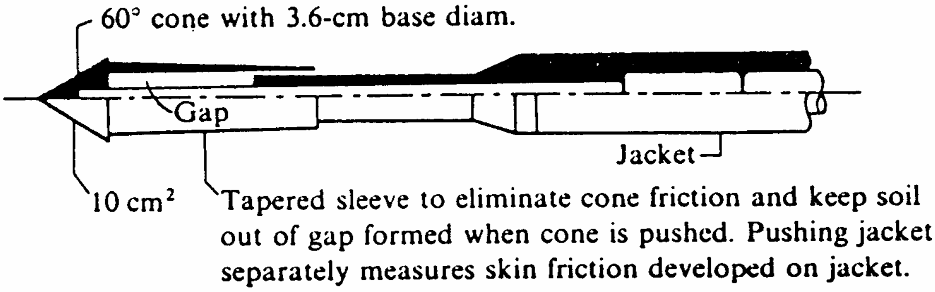


Figure 10.1 - Dutch Cone Penetrometer

Advantages

Some of the advantages of the Dutch cone penetration testing method are that it :

- is very rapid - particularly when electronic data acquisition equipment is used to record the tip pressure and/or side resistances
- may allow a nearly continuous record of resistance in the stratum of interest
- it is useful in very soft soils where recovery of *undisturbed* samples would be very difficult
- allows a number of correlations between cone resistance and the desired engineering property

Disadvantages

Some of the major disadvantages of the Dutch cone penetration testing method are that :

- this method is only applicable to fine-grained deposits (clay, silt, fine sands) where the material does not have massive resistance to cone penetration
- interpretation of soil type producing the cone resistance requires either :
    - considerable experience, or
    - recovery of samples for correlation testing.

10.2.3 Standard Penetration Test (SPT)

Dynamic Testing

The *Standard Penetration Test (SPT)* is the most widely used for borehole dynamic tests. This test has a dual function : both penetration testing and sampling. It therefore makes it possible to identify changes in the soil by two independent methods and for this reason it is such a useful tool in exploration.

Testing Method

The *SPT* is generally used to determine the bearing capacity of sands or gravels and is conducted with a split spoon sampler (a sample tube which can be split open longitudinally after sampling) with internal and external diameters of 35 and 50 mm respectively. The sampler is lowered down the borehole until it rests on the layer of cohesionless soil to be tested. It is then driven into the soil for a length of 450 mm by means of a 65 kg hammer free falling 760 mm for each blow. The number of blows required to drive the last 300 mm is recorded and this figure is designated as the *N* value of the soil (the first 150 mm of driving is ignored because of possible loose soil in the bottom of the borehole from the boring operations). After the tube has been removed from the borehole it can be opened and its contents examined.

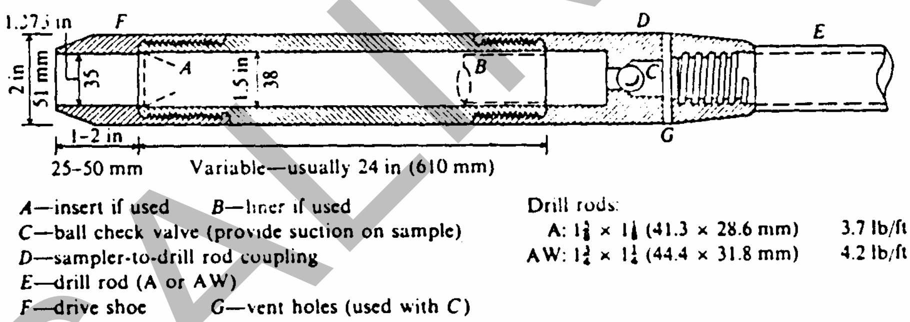


Figure 10.2 - Standard Penetration Test Split Spoon Sampler

Testing below Water Table

The standard penetration test, when properly conducted and corrected for overburden pressure, is the only economical means of assessing bearing capacity of sands and gravels beneath the water table. However, when carrying out the SPT in sand below the ground water table level, it is important to keep the water level in the bore up to approximately the same level as in the ground throughout the drilling operation. A differential head will cause an upwards hydraulic gradient which could loosen the sand in the bottom of the borehole.

Advantages

Some of the advantages of the Standard Penetration Test are that it :

- is extremely economical in terms of cost per unit of information
- allows both penetration testing and sampling to be carried out
- allows correlation of material properties to a large SPT data base which is continually expanding
- has testing equipment which has a long service life
- readily allows other tests to supplement the SPT when the borings indicate that more refinement in sample and data collection is required.

Disadvantages

Some of the major disadvantages of the Standard Penetration Test are that :

- the test is difficult to reproduce and is subject to many errors in practice, including the skill of the operator
- it is not reliable in gravel and soils containing large gravels. In loose gravels the split spoon tends to slide into voids giving low penetration resistance. The split spoon also tends to rotate the round pebbles as it penetrates into voids, thus producing low readings. If the spoon is blocked by gravel, excessively large resistance to driving can be expected.

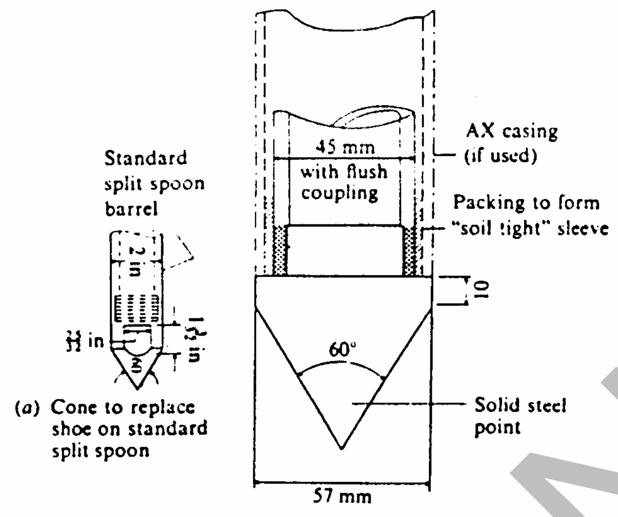
10.2.4 Dynamic Cone Penetration Test

Dynamic Testing

The *Dynamic Cone Penetration Test* is another type of commonly used non-borehole dynamic test. This test is sometimes used as a substitute for the standard penetration test, particularly in hard rocky strata where the split spoon sampler is likely to be damaged. When used for other soil types, such a penetrometer should be correlated against the standard penetration test if maximum benefit is to be obtained from the results.

Testing Method

In this test a cone is driven into the ground in the same way as the SPT spoon is driven. But unlike the SPT, there is no preboring involved. There are many varieties of this type of penetrometer but all consist basically of a conical point 35 to 60 mm diameter with an apex angle of 45° to 60°. The dynamic cone penetration test can be used with or without bentonite (mud) slurry, but when the depth of investigation is more than 6 m, use of bentonite or mud slurry is recommended as otherwise friction on the rods would be very high. Data from the dynamic cone penetration test is plotted as a curve of penetration resistance N, the number of blows per 300 mm of penetration, versus depth.



(*a*) Cone to replace shoe on standard split spoon

(*b*) Cone attached to drill rods.

Figure 10.3 - Dynamic Cone Penetrometer

Advantages over Boring and SPT

Some of the advantages of the dynamic cone penetration testing method are that it :

- is faster and more economical than boring or SPT. It is primarily useful in mapping of soil strata during the early stages of explorations when the number of borings is normally limited. During detailed investigation some geotechnical engineers may prefer to substitute a single borehole by a number of dynamic cone tests without an increase in cost and obtain more relevant information between the borings.

- gives continuous penetration of strata being tested, often revealing the presence of strata which are not recovered or observed in sampling operations.

Disadvantages

Some of the major disadvantages of the dynamic cone penetration testing method are that :

- either no samples or only wash samples are obtained from it and therefore strata cannot be definitely identified by penetration alone

- presence of gravels or boulders within the soil strata can give misleading results. Consequently interpretation of results obtained from the dynamic cone penetration testing requires considerable experience, particularly in those areas in which correlations between the penetration resistance and engineering properties of soils penetrated are to be developed.

10.2.5 Determination of Soil Parameters

Soil Strength Assessment

Sufficient information has been built up on the use of the standard penetration test and the Dutch cone penetrometer test for these tests to directly provide a means of assessing the relative strength of soil deposits of similar soil particles. Because of the widespread use of these two tests, many charts have been established which relate approximately the blow count or cone resistance to the various soil properties, as follows :

Standard Penetration Test

Granular Soils

- relative density
- soil settlement and elastic modulus
- angle of internal friction

Cohesive Soils

- consistency (rather variable)

Dutch Cone Penetrometer

- pile bearing capacity
- pile side wall friction
- elastic properties of strata for settlement analysis
- cohesion (or consistency) values for clay

Interpretation of Penetration Test Results

In a localised area a relationship between penetration resistance and degree of weathering of weathered rock can often be obtained. In order to use the above relationships it is necessary to have a good knowledge judgement based on past experience. The penetration tests should never be considered in isolation to obtain these soil properties but must be carried out in conjunction with core drilling and sampling. Without sound judgement the use of charts to obtain the above soil properties can be misleading and dangerous practice.

10.3 VANE TEST

Application

This is a useful test for determining the undisturbed and remoulded undrained strengths of soft, normally consolidated clays. As a general rule, the vane test should only be carried out in fine grained soils (silts and clays) and in particular in soft soils but it should never be used to assess the strength of granular soils. This test will give a more reliable assessment of the strength of soft clays than the standard penetration test.

Testing Method

The vane test is suitable for testing soils below the bottom of a borehole at great depths with a minimum disturbance. Field vanes employ two crossed blades attached to a vertical rod. Typical vane diameters are 50, 65, and 75 mm with lengths of 3 to 5 diameters. The vane is forced into the soil so its top is 2 diameters below the bottom of the borehole. Rotation of the vane shears the soil on a cylindrical surface. The torque required to initiate shear is measured, and often the increasing torsional strain is indicated as a function of torque. After failure, a second torque is made after several revolutions, to measure the soils remoulded strength.

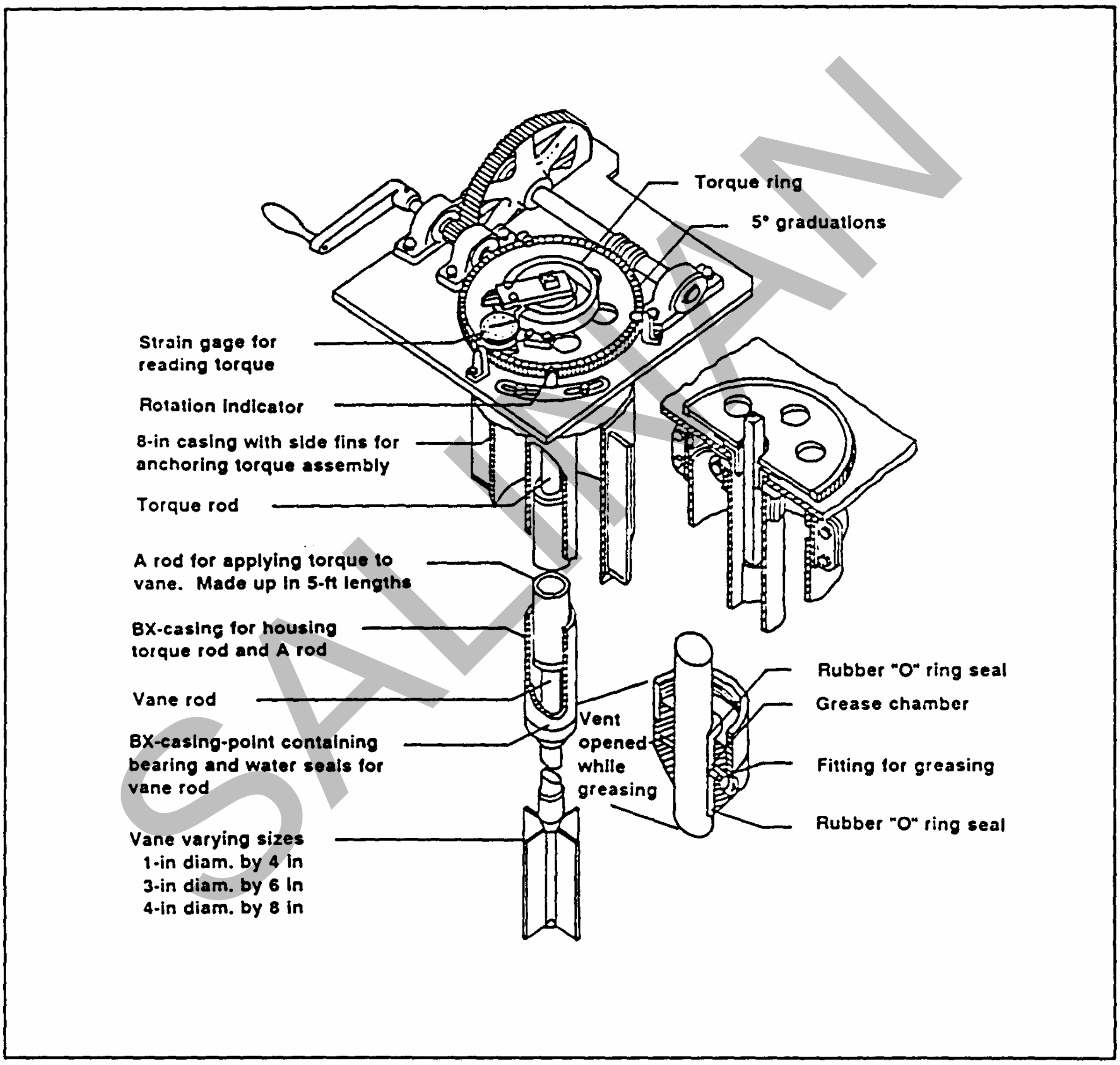


Figure 10.4 - Vane Test Apparatus

The vane test torque head is usually mounted at the top of the rods. This is standard practice for most site investigation work but, for deep bores, it is now possible to use apparatus in which the torque motor is mounted down near the vane, in order to remove the whip in the rods. Because of this development the vane has largely superseded the standard penetration test for deep testing. The SPT has the disadvantage that the load must always be applied at the top of the rods so that some of the energy from a blow must be dissipated in them. This energy loss becomes more significant the deeper the bore, so that the test results become more unreliable.

A type of vane is also manufactured which can be advanced independent of a borehole with the vane in a retracted position in its own protective outer casing. When in position, the vane is extended out of the casing and the test carried out.

The vane test can also be carried out in the walls of a test pit.

Interpretation

The cylindrical shear surface in the vane test does not resemble soil failure in real problems nor in laboratory tests. Neither do strains that develop before failure. Therefore, the results of vane tests do not always agree with other shear tests. Although empirical corrections have been proposed, the vane strengths should be used with caution in highly plastic or very sensitive soils, where experience shows that vane strengths exceed other test results.

10.4 WATER TABLE

Importance of Water Table Measurements

Measurements of ground water table level should be taken as a standard procedure in any excavation or boring which penetrates below the water table. However, the bore or pit must be left open for long enough to allow the water table to settle to an equilibrium position prior to measurements being recorded. In sandy soils a few hours is adequate, but in clays a week or more is required.

Borehole Observations

In most cases where normal groundwater conditions are encountered they can be investigated during boring. The water level should be measured at regular intervals during the advancement and after completion of each borehole.

During each boring, field records should be made of all observations related to groundwater such as cahnge in colour and rate of flow, partial or total loss of water, and first appearance of artesian conditions.

All information related to groundwater should be recorded on the boring log, along with the depth of the borehole and depth of casing at the time of observation.

Groundwater observations made at the time of boring are not representative in clay and other fine-grained soils because of the low permeability of these materials and the longer periods of time required before the water level in such a borehole reaches equilibrium.

Water Level Probe

In order to detect the ground water level accurately, a slender electric probe is necessary. It consists of two insulated wires embedded in a weighted sleeve that will fit inside the borehole. The wire ends, uninsulated, extend 1 to 2 mm below the notch in the sleeve. When the wires touch water, there is sufficient conductivity so that the current can be indicated by a milliammeter or activates a buzzer. A measuring tape attached to the probe indicates the depth below the top of the borehole at which the water level occurs.

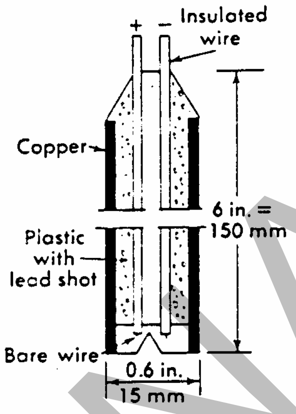


Figure 10.5 - Water Level Probe

10.5 FIELD LOAD TEST

10.5.1 General

Measurement of Elastic Modulus

Field load tests are carried out to directly measure the elastic modulus of the soil. The first cycle of loading usually produces higher deformation due to bedding in displacement and subsequent loadings give a more accurate measure of the vertical subgrade modulus. These test results can also be used to assess the values of lateral subgrade modulus using elastic theory. If such tests can be carried out in a rapid manner (loading time less than 0.2 seconds) the test can be used to determine the dynamic modulus or elasticity required in seismic foundation designs. However, as most equipment permits only a much slower rate of loading, such tests tend to under-estimate the value of the dynamic modulus. (Hence either a factor of the order of 2 should be applied to the results of a load test to obtains the correct dynamic modulus, or alternatively the dynamic modulus should be estimated using seismic methods).

Bearing Capacity

In some cases a field load test is carried out to simulate more accurately the load to be applied to the ground by a structure. Some form of kentledge is used, such as calibrator weights, scrap metal, large stones, soil or tanks filled with water. Such tests are generally conducted to measure ground deformation at the foundation location but can be contrived to produce a bearing capacity failure of the ground if the test site is not to be used subsequently.

10.5.2 Plate Bearing Test

The most common field load test is the plate bearing test which is a small scale test generally carried out with either a square plate (usually 300 mm square) or a around plate up to 750 mm diameter. The 750 mm diameter plate can also be used to determine a modulus of subgrade reaction at a deflection of 1.3 mm (0.05 inches). The depth of influence of this test is only of the order of 1½ to 2 times the plate diameter and so its use is limited unless the material is known to be uniform to considerable depth.

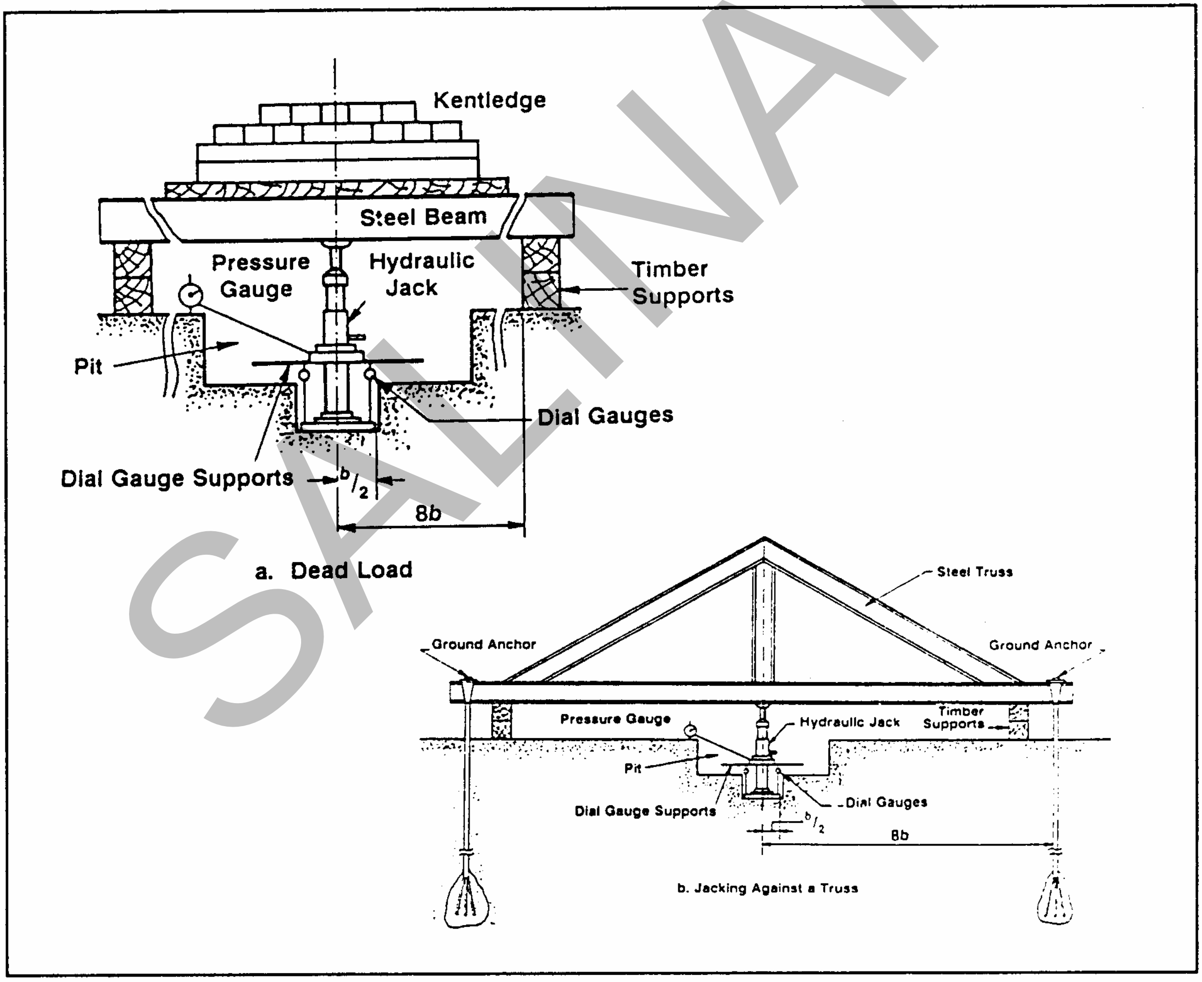


Figure 10.6 - Plate Bearing Test

10.5.3 Pile Load Test

Another form of large scale field test is the pile load test. A vertical test load is applied to the pile usually via jacks and several cycles of loading at controlled loading rate applied. The jack is generally loaded against reaction piles adjacent to the first pile. The definition of *failure* in such tests is not always clear. Many tests are mounted with reactions inadequate to produce shear failure in the soil, in which case they function only as proof tests to some particular load.

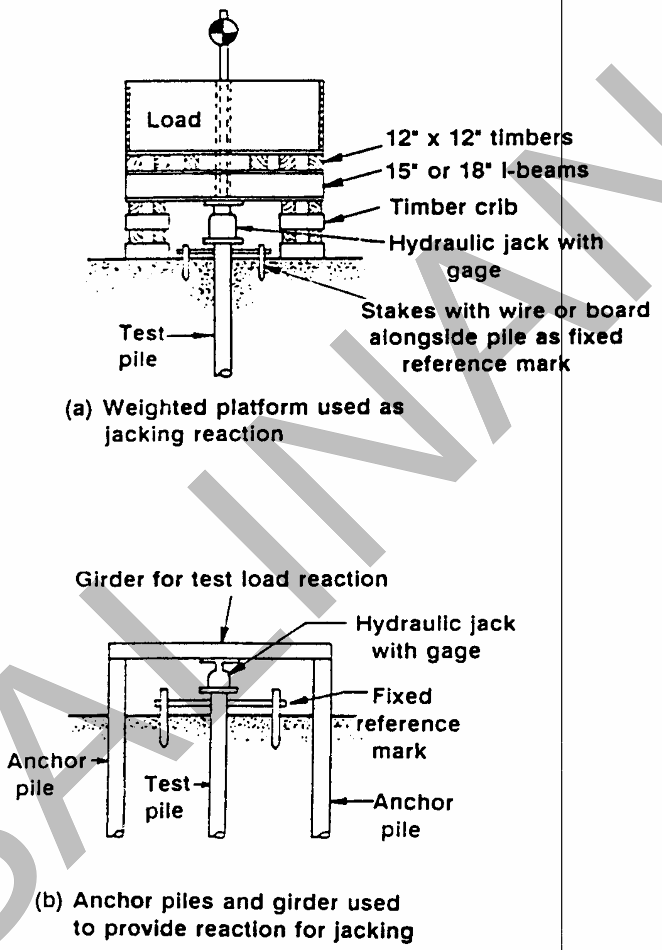
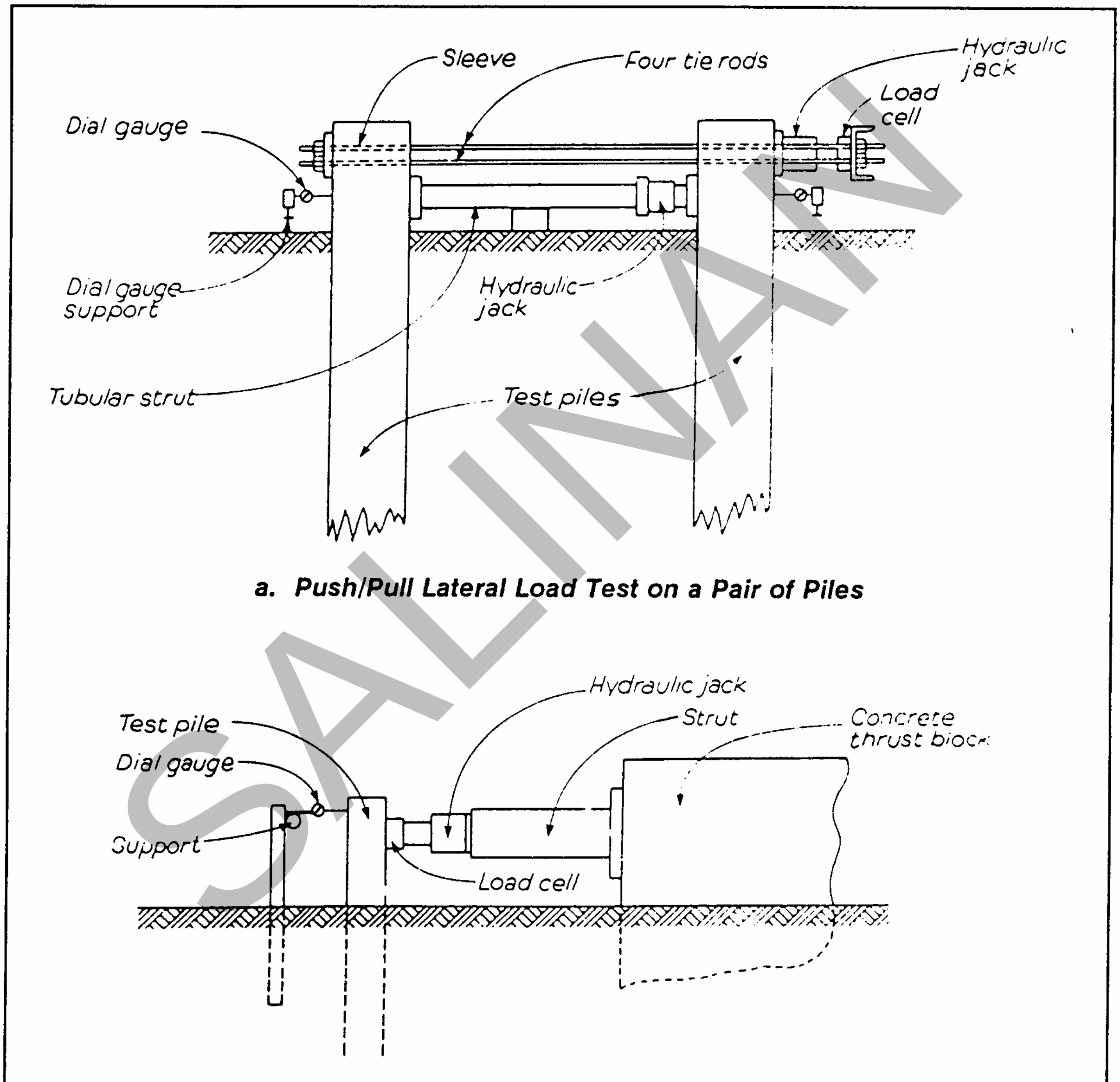


Figure 10.7 - Pile Load Test

10.5.4 Lateral Pile Test

A further test of particular relevance to the earthquake behaviour of bridge structures is the lateral pile test in which a pile is jacked sideways by reaction off an adjacent pile. The piles should be placed sufficiently far apart so as not to obtain significant interaction between movements of each pile, and hence a horizontal beam is frequently inserted between the piles and the jack reacts against one of the pile heads and the beam to the other pile head. Lateral deformations of the test pile are measured in order to obtain a load deformation curve for the top of the pile.



a. Push/Pull Lateral Load Test on a Pair of Piles

b. Lateral Load Test on a Single Pile

Figure 10.8 - Lateral Pile Test

10.6 PRESSUREMETER TESTS

The most common test of this type is the Menard pressuremeter which measures the lateral deformation of the soil around a limited depth of borehole (usually 1 m) when subjected to controlled internal pressure. Successive tests along the length of a bore provide the modulus (E) and the limiting pressure (P_c) representing general failure of the soil. The borehole must be drilled to a close tolerance on diameter to suit the pressuremeter. Identification of the soil type and selection of the appropriate test to be conducted must be made. This test has the advantage over laboratory tests in that sample disturbance is avoided and a larger mass of soil is tested at each depth.

The Menard pressuremeter consists of the probe, which is inserted into the borehole, a surface-stationed control unit that controls and monitors the probe's pressure and volume changes, and tubing that connects the control unit and probe. The pressuremeter probe consists of three cells : top guard cell, test cell, and a bottom guard cell. The top and bottom guard cells are expanded to reduce end-condition effects on the test cell which is used to obtain the volume versus cell pressure relationship used in data reduction. Water is typically used in the test cell to measure the volume changes that occur as pressure increases. Bottled compressed gas is used to pressurise the test and guard cells.

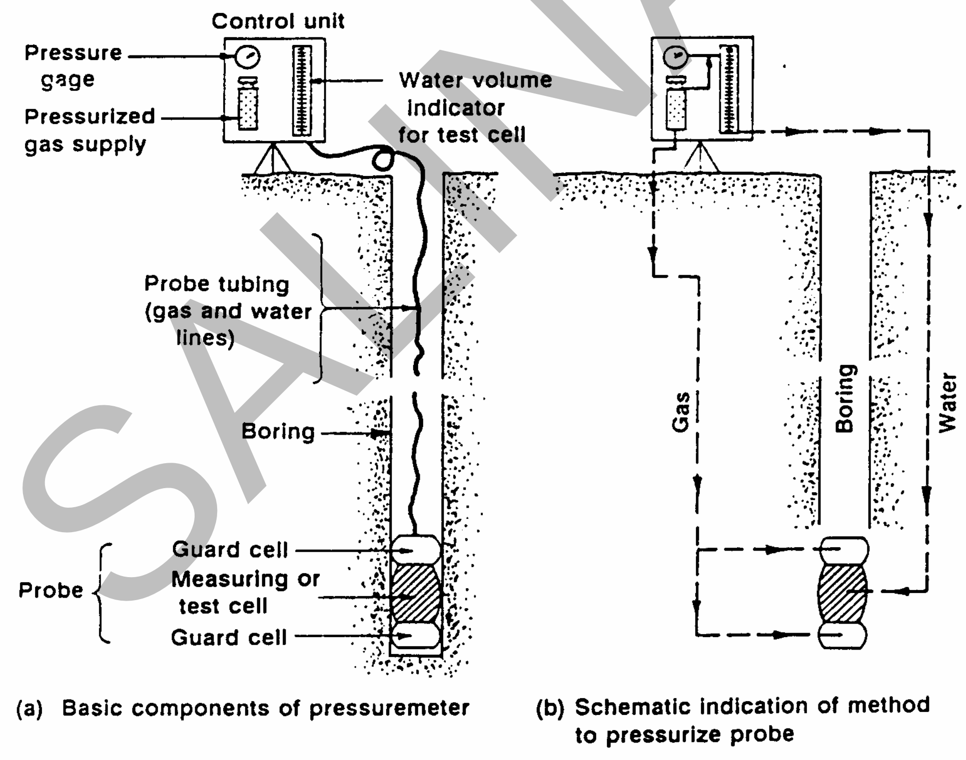


Figure 10.9 - Menard Pressuremeter

The pressuremeter test is not a trivial task, and therefore is not commonly used, as fairly high pressures are involved and calibrations for pressure and volume losses must be made to correct the pressure-volume data taken during the test. The test can only be performed in soils where the borehole can be shaped and will stand open until the probe is inserted. Another factor of concern is that the soil tends to expand in the cavity when the hole is opened so that the test often has considerable disturbance effects included. The pressuremeter seems to have best applications in relatively fined grained sedimentary deposits.

The pressuremeter modulus ***E*** is a lateral value and, unless the soil is isotropic, is different from the vertical value which is needed for bearing capacity and settlement studies and therefore has more application for laterally loaded piles and caissons.

10.7 FIELD UNCONFINED COMPRESSION TESTS

The unconfined compression test may be carried out in the field using reasonably simple equipment to assess the strength of stiff clays, weakly cemented sand or weathered rock. The test can also be performed on soft clays provided suitable samples can be obtained without disturbance, but generally for these soils more accurate results will be obtained using the in-situ shear vane test.

Because of its simple nature, field unconfined compression testing equipment does not give very accurate results and for this reason such test results are generally regarded as being an aid to soil description and classification rather than an accurate measure of soil strength.

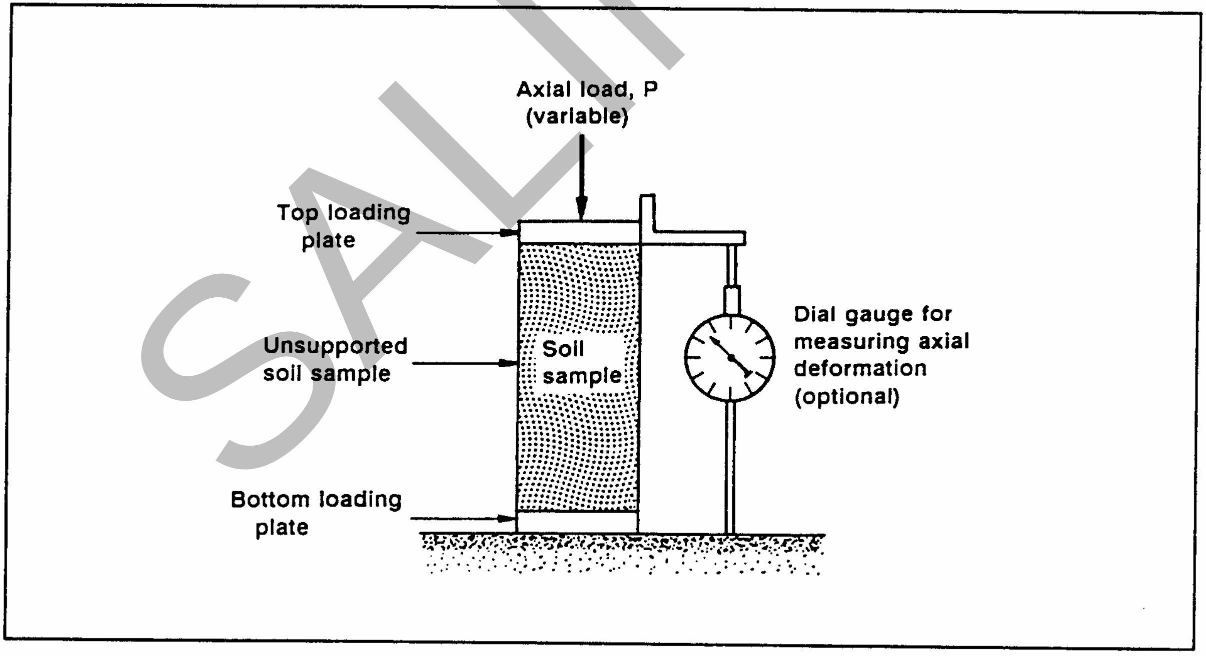


Figure 10.10 - Unconfined Compression Test

10.8 IN-SITU SOIL DENSITY TESTS

Common methods of measurement for in-situ soil density involve replacement of the soil excavated from a density hole with oil, sand or seed. Oil should not be used for highly permeable or porous soil. Another common method of measurement of the volume of an excavated density hole is the balloon densometer method but this is not very satisfactory if the density hole cannot be made smooth.

10.9 REFERENCES

Refer to Section 9 of this manual for list of *Indonesian Language* and *English Language* references.

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DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
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BRIDGE INVESTIGATION MANUAL

SECTION 11

LABORATORY TESTING

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11. LABORATORY TESTING

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11. LABORATORY TESTING

11.1 INTRODUCTION

The principal types of laboratory testing available to determine soil parameters are :

- shear box test (direct shear)
- triaxial test
- unconfined compression tests
- one-dimensional consolidation test
- laboratory shear vane test
- compaction tests
- soil classification tests.

11.2 SHEAR BOX TEST (DIRECT SHEAR)

11.2.1 General

The primary objective of soil strength measurement is to determine the failure envelope, which is the relationship between τ and σ. One of the most widely used methods for the measurement of shear strength of soils is the *direct shear test*. Sample preparation and test operation are simple for most soils making the test attractive for routine work.

11.2.2 Testing Method

A sample of soil is placed in a rectangular box, the top half of which can slide over the bottom half. The lid of the box is free to move vertically, and to it is applied the *normal load*, ***Q***. A *shearing force*, ***F***, is applied to the top half of the box, shearing the sample along plane *X-X*. In practice, the top and bottom of the box may be either porous plates to permit changes in the water content of the sample or projecting vanes to help develop a uniform distribution of stress on the failure surface.

11.2.3 Advantages and Disadvantages

Advantages

The advantages of the direct shear test are :

- the sample preparation and test operation are simple for most soils, which makes the test attractive for routine work

- the direct shear test utilises a relatively thin sample which consolidates rapidly under load when such consolidation is required.

Disadvantages

Inherent shortcomings limit the reliability of the direct shear test results as follows :

- there is an unequal distribution of strains over the shear surface; the strain is more at the edges and less at the centre. The result is progressive failure. In materials with highly developed structures, such as flocculent clays and cemented or very loose non-cohesive soils, the strength indicated by the test will often be too low.

- the soil is forced to shear on a predetermined plane, which is not necessarily the weakest one. The strength given by the test, therefore, may be too high.

- it is difficult to control the drainage or changes in water content during the test, which limits its usefulness in wet soils.

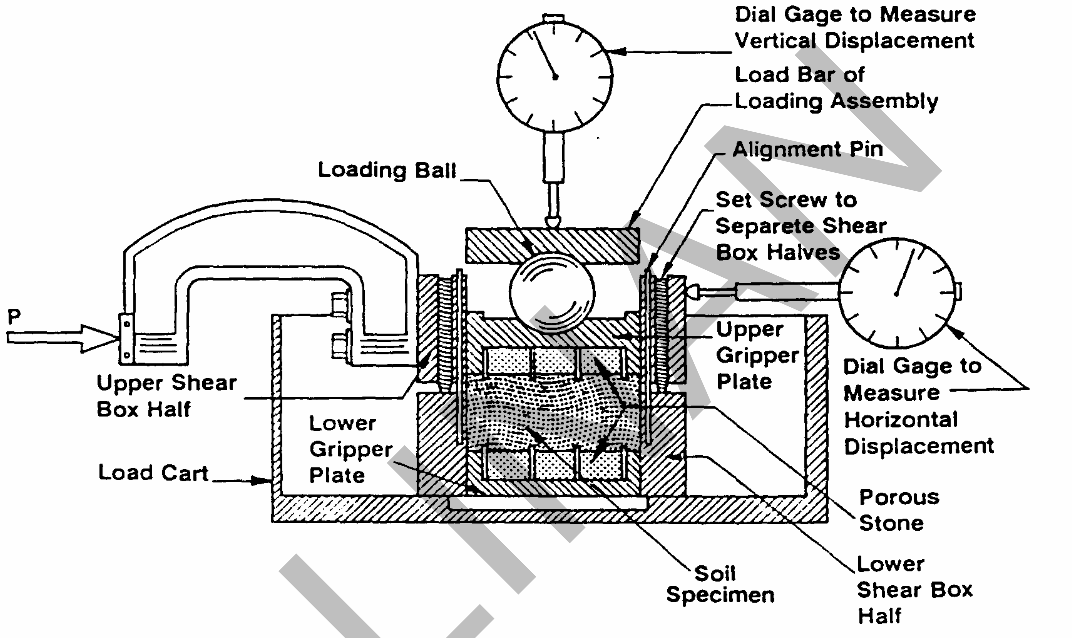


Figure 11.1 - Direct Shear Test

11.2.4 Dynamic Shear Modulus

Shear box tests carried out at rapid loading rates can be used to determined the dynamic shear modulus of a soil. (This parameter is required if a site response analysis is to be carried out for a major bridge structure). An alternative method of determining dynamic shear modulus is by means of the dynamic torsion test developed at the University of Auckland (Reference 11.1).

11.2.5 Other Parameters

The shear box test can also be used to provide other useful parameters as follows :

- **Shear Strength**

  Shear box tests carried out at normal loading rates (static shear box test) can provide similar shear strength parameters to triaxial tests carried out at the same loading rate. Generally there is a greater scatter of results than with triaxial tests, but for granular materials samples can be more easily prepared with less disturbance.

- **Residual Strength**

  Static shear box tests are also to determine the residual strength of over consolidated clays or the joint strength of shattered rock (Reference 11.2) as the shearing takes place along a defined failure plane.

- **Volume Characteristics**

  Cyclic shear box tests can be used to determine the volume characteristics of sand and hence their liquefaction potential (Reference 11.2).

11.3 TRIAXIAL TEST

11.3.1 General

The triaxial test is the most commonly used testing method for the determination of soil shear strength parameters. It is usually the preferred test due to its simplicity and commercial availability. It is the most reliable shear test for routine soils testing and is considered to provide the best soil parameters and stress strain data (for *stress-strain modulus* E_s, *Poisson's ratio* μ, and *shear modulus* G_s). However, very careful sample preparation is required for triaxial test specimens.

The two forms of triaxial test are :

- **Static Triaxial Test**

  Static or slow loading rate triaxial tests are commonly used to determine volume changes of a soil during shearing, soil strength parameters (total stress parameters) and/or effective stress parameters (with pre-pressure measurements) and consolidation characteristics.

- **Dynamic Triaxial Test**

  The dynamic triaxial test which requires loading equipment beyond the sophistication of commercial apparatus which is normally available. The test may be either stress or strain controlled (Reference 11.3) and is often used to determine the elastic modulus of a soil, or assess the number of cycles to liquefaction for a sand.

11.3.2 Testing Method

Normal Triaxial Test

A cylindrical sample is used for the triaxial test with a diameter ranging from 35 to 75 mm, and a length ranging from 2 to 3 times the sample diameter. The sample is encased in a rubber membrane, with rigid caps or pistons both ends. It is placed inside a closed chamber and subjected to a confining pressure σ_3 on all sides by air or water pressure. An axial stress σ_1

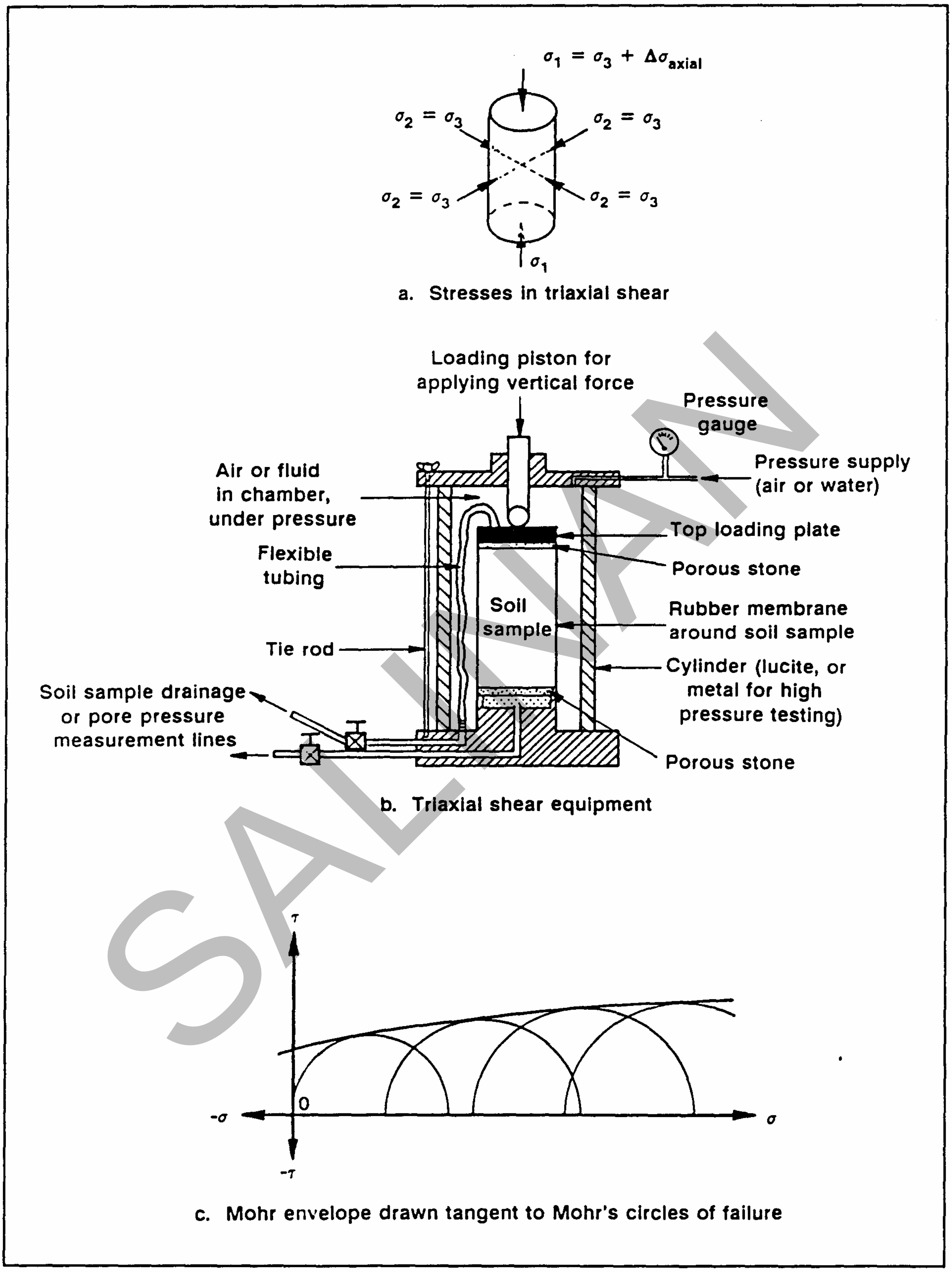


Figure 11.2 - Triaxial Test

is applied to the end of the sample by a piston. Either the axial stress can be increased or the confining pressure decreased until the sample fails in shear along a diagonal plane or a number of planes. The Mohr circles of failure stresses for a series of such tests, using different values of σ_3, are plotted and the Mohr envelope drawn tangent to them.

Triaxial Extension Test

An alternative procedure for the triaxial test is to hold the axial stress constant and increase the confining pressure until the sample bulges upward in the axial direction. In this form, the triaxial extension test, the confining pressure is $\sigma_1=\sigma_2$ and the axial stress is σ_3. This procedure is used to simulate the effect of a lateral thrust on a mass of soil. The Mohr envelope is similar to that for the compression test in a homogeneous isotropic soil; in stratified materials it is often different.

Triaxial Unconfined Compression Test

A special case of the triaxial test is the unconfined compression test in which $\sigma_3=0$.

11.3.3 Drainage Conditions

If a triaxial test is used solely to measure the soil strength during undrained shear, there is no need to measure the pore pressures developed during the test. However, measurement of pore pressures permits a determination of the effective stresses during undrained loading and leads to an understanding of the relationship between undrained and drained strength.

Three drainage conditions are commonly available during triaxial testing. The triaxial tests under different drainage conditions are :

- **Unconsolidated Undrained Test (UU)**
  Applicability : assessment of short term slope stability

  In this test the sample may (or may not) be confined by a consolidation pressure as is simply tested to failure in compression of shear. This test is commonly termed an undrained test for strength s_u. The unconfined compression test is a UU test with a failure compressive strength q_u. This test gives ϕ = *zero* for saturated soils and a range from *0* to ϕ' for others - depending on water content. Any sample confining pressure tends to give compressive failure stresses larger than q_u.

- **Consolidated Undrained Test**
  Applicability : assessment of long term slope stability

  In this test the sample is consolidated under a specified pressure prior to failure in compression or shear.

  - **without pore pressure measurements (CU)**

    The range in ϕ without pore pressure measurements is from *0* to ϕ' - values slightly larger than zero are usually obtained.

- **with pore pressure measurements ($\bar{C}\bar{U}$)**

  If pore pressure is measured the effective stress parameters ***c′*** and ***ϕ′*** can be obtained.

- **Drained Test (CD)**
  Applicability : assessment of long term slope stability

  In this test the sample is consolidated as for the consolidated undrained test but during testing to failure the test is done so slowly that excess pore pressures from the shear strains are not sufficiently large to significantly affect the effective stress soil parameters that are directly obtained.

11.3.4 Advantages and Disadvantages

Advantages

The important advantages of the triaxial test are the relatively uniform stress distribution on the failure plane and the freedom of the soil to fail on the weakest surface. Furthermore, water can be drained from the soil or forced through the soil during the test to simulate actual conditions in the ground. Sample preparation is simple, and small-diameter cylindrical samples can be used.

Disadvantages

The main disadvantage of the triaxial test is the elaborate equipment required, including sample membranes, compressed air or water-pressure equipment, the triaxial cell itself, and auxiliary devices to measure the volume change of soil during testing. The normal triaxial test utilises rigid end caps. These restrain the shear and cause stress concentrations that change conditions in failure. It is limited to values of $\sigma_2 = \sigma_3$ and $\sigma_2 = \sigma_1$ in compression and extension, respectively.

The triaxial test does not have the advantages of the shear box test and torsion test when applied to seismic work.

11.4 UNCONFINED COMPRESSION TESTS

Laboratory unconfined compression tests can give more accurate results than field tests if more sophisticated loading equipment is available. For soft, saturated normally consolidated clay, the unconfined compression test can be used as a substitute for the triaxial test in obtaining the soil cohesion (assuming $\phi = 0$). For soils for which ϕ cannot be assumed to be zero, the unconfined compression test cannot be used to determine shear strength parameters.

11.5 ONE-DIMENSIONAL CONSOLIDATION TEST

11.5.1 Consolidation Mechanism

The compressibility of soils arises from the relatively large percentage of voids in soils. The stresses encountered in most engineering works are far too small to produce significant changes in the volume of solid particles. The volume change in a soil is almost entirely the result of a reduction of the void volume. This occurs as the applied stresses distort or break down the existing soil-skeleton structure locally, forcing the particles to from a denser structure with a lower void ratio.

The compressibility of a soil is the amount of volume compression that the soil undergoes per unit increment in stress. The amount of compression may be expressed in terms of void ratio or as a percentage of the initial volume.

11.5.2 Testing Method

Description

The soil specimen is placed inside a metal ring. Pressure is applied to the specimen through the loading head, and the porous stones allow free escape of the porewater as the voids in the soil are compressed. A dial indicator is used to measure the downward movement of the loading head; this allows the calculation of the volume change of the soil.

In normal consolidation tests the pressure is doubled every 24 hours. The long time interval between load increments is necessary because the drainage of water from the voids does not occur instantaneously but requires considerable time.

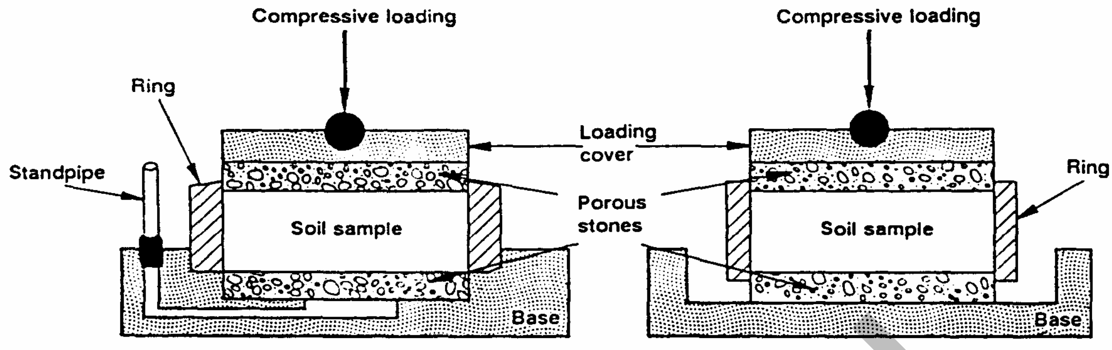


Figure 11.3 - One-Dimensional Consolidation Test

Results

The consolidation test gives the empirical relationship between the pressure and the volume change that is called *compressibility*. Compressibility m_v, is defined as the volume change per unit volume for a unit increment of stress as applied in the consolidation test. The value of m_v is not constant throughout the entire range of stress, although for small stress increments it is often taken as constant.

It should be noted that this test produces compression in the vertical direction only, while the lateral dimensions remain unchanged. Under such a condition, the vertical stress, σ_1, and the horizontal stress, $\sigma_2 = \sigma_3$, are found to maintain a constant ratio K_o.

The results of the consolidation test are usually presented in the form of a pressure versus void ratio curve. Clays usually show far greater compressibility than silts and sands.

11.5.3 Applications

a. Primary Consolidation

The one-dimensional consolidation test is used on cohesive soils to determine the likely long term consolidation settlement of a stratum of the soil under an applied load (for example, a bridge foundation or approach embankment). An estimate of the rate of consolidation can also be made on the basis of coefficients of consolidation derived from the one-dimensional consolidation test under a similar increment of applied load.

b. Secondary Compression

The same test can be used to obtain an estimate of the likely settlement due to secondary compression which will occur under a given load increment after the primary consolidation is complete and excess pore pressure is dissipated.

Secondary compression continues very slowly at an ever-increasing rate indefinitely. It appears to be the result of plastic readjustment of the soil grains to the new stress, of progressive fracture of the interparticle bonds, and progressive fracture of the particles themselves.

For organic soils of low to moderate compressibility, secondary compression is seldom important. It can be a major part of the compression of highly compressible clays, highly micaceous soils, fills of broken rock, and organic materials.

Continuing settlements due to secondary compression can be very significant beneath bridge embankments founded over soft alluvial or organic soils.

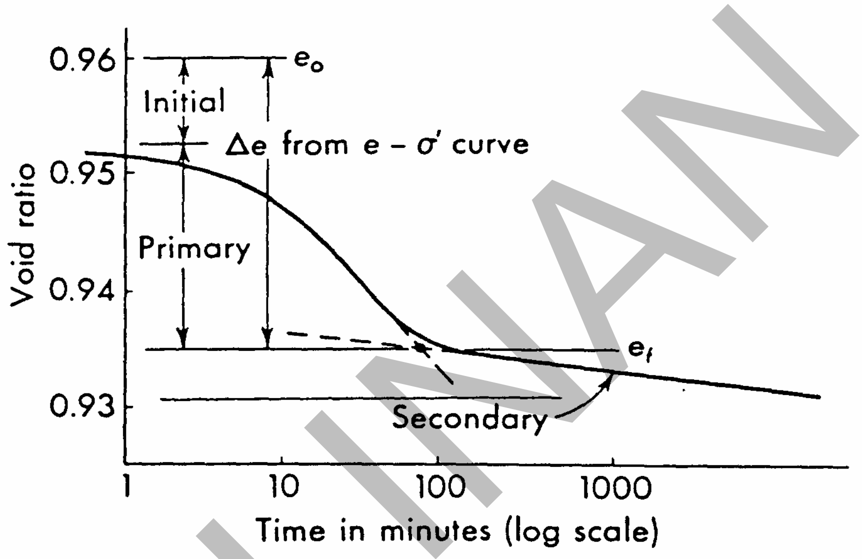


Figure 11.4 - Secondary Compression

c. Coefficient of Permeability

From consolidation test data it is also possible to make an estimate of the coefficient of permeability of a soil.

11.6 LABORATORY SHEAR VANE TEST

This test can be used as an alternative to the unconfined compression test or triaxial test to determine the cohesion of soft normally consolidated clay samples. In the laboratory a miniature vane (usually ½ inch diameter) is used. Sensitivity of the clay (that is, the ratio of undisturbed to remoulded strength) can be readily measured in this way and this will give an important indication of the likelihood slumping under earthquake conditions of embankments founded on soft clay soils.

11.7 COMPACTION TESTS

11.7.1 Compaction Curves

Compaction curves are used as a control for the placement of cohesive or non-cohesive fill and the measured in-situ soil density is compared with the compaction curves. (A well compacted fill will be much less prone to severe damage under earthquake than a poorly compacted fill. Also if the ground beneath the fill fails during the earthquake then the well compacted fill is more likely to settle as a unit than the weaker, less compacted fill).

For fine grained cohesive soils compaction curves are usually obtained using either a 2.5 kg or 4.5 kg falling hammer (for example, Tests 12 and 13, BS 1377:1975). For uniform granular soils the falling hammer method is not very effective in compacting the soil and vibrating hammer method of compaction can be used as an alternative (for example, Test 14, BS 1377:1975).

The measured in-situ density and moisture content of either natural or compacted soil can then be compared to one of these curves.

11.7.2 Relative Density Test

A further test commonly carried out on granular soils is the relative density test in which the in-situ density is compared with the maximum and minimum densities which can be achieved with the same soil (refer US Bureau of Reclamation Earth Manual, Reference 11.4). This test is prone to large experimental errors and it is difficult to ensure accurate results. However, the relative density of sand is an important parameter in the assessment of liquefaction potential.

The major reason for using relative density is that undisturbed sampling of in-situ non-cohesive sands and gravels is nearly impossible, and, as a consequence penetrometer testing is widely used. A large data base presently exists (with considerable scatter) relating penetration tests to relative density.

11.8 SOIL CLASSIFICATION TESTS

11.8.1 General

In-situ moisture content, in-situ dry density, liquid limit, plastic limit and specific gravity test are common laboratory tests often carried out in conjunction with triaxial tests, consolidation tests or other more specialised tests. These tests are carried out for classification purposes, that is, similar soil types in various boreholes can be identified by classification tests and hence related to the more specialised tests on similar soils in another borehole.

Generally on every undisturbed soil sample recovered during a site investigation the in-situ moisture content should be measured in the laboratory and also the in-situ dry density of the sample in the tube should be measured as a standard procedure.

11.8.2 Unified Soil Classification System

Description

The *Unified Soil Classification System* shown in Table 11.1 and Table 11.2 is the most commonly one used in foundation design. The soils are organised into the following size groups : gravels (G), sands (S), inorganic silts and fine sands (M), inorganic clays (C), and organic silts and clays (O). Each group is then divided into subgroups according to their significant index properties. The gravels and sands with little or no fine materials are subdivided according to their size-distribution properties into well graded (GW and SW) or uniform (GP and SP). If the soil contains more than 12% fines, their properties must be taken into account. Since the fine fraction in soils may have substantial influence on soil behaviour, the gravels and sands have two other subdivisions. Those with fine fraction that serves as good binder material (mostly silts) are classed GM or SM. If the fines contain plastic clays, the soils are classed as GC or SC.

For the fines the most important index property is the Atterberg limits, which is used to subdivide the clays and silts. The liquid limit and plasticity index are plotted on a plasticity chart. The A-line on the plasticity chart is the arbitrary boundary between the inorganic clays (CL and CH) which are above this line and the inorganic clays (ML, MH, OL, and OH) which are below. It has been found that samples of soils of similar geologic origin and composition usually yield points that fall on a line parallel to the A-line. The clays and silts are further divided into those of high and low compressibility according to liquid limit. This is based on the empirical observation that the compressibility of a soil increases with liquid limit. Those with a liquid limit in excess of 50% are classified as high compressibility (MH, CH). If the liquid limit is less than 50%, they are classified as low compressibility (ML, CL).

The organic clays may be distinguished from the inorganic silts by their characteristic odour and black colour. Also oven-drying greatly alters the Atterberg limits of organic soils whereas its effect on inorganic soils is much smaller.

Limitations

It should be emphasised that natural soils do not consist of distinct groups but form a broad spectrum. Thus the dividing lines used in classification are necessarily arbitrary. Secondly, all soil classifications are based on their index properties. They can, at best, serve as nomenclature for describing soils and provide some indication as to the significant engineering properties. None of the soil-classification systems, however elaborate, should be used as a measure of the soil's engineering properties (such as strength or compressibility) for the obvious reason hat such properties are not measured in the classification system. Furthermore, the soil properties that are important depend on the type of problem to be solved. It is thus obvious that a rigid classification system cannot hope to include in it all the soil properties that may be needed to solve the many diverse problems in soil mechanics.

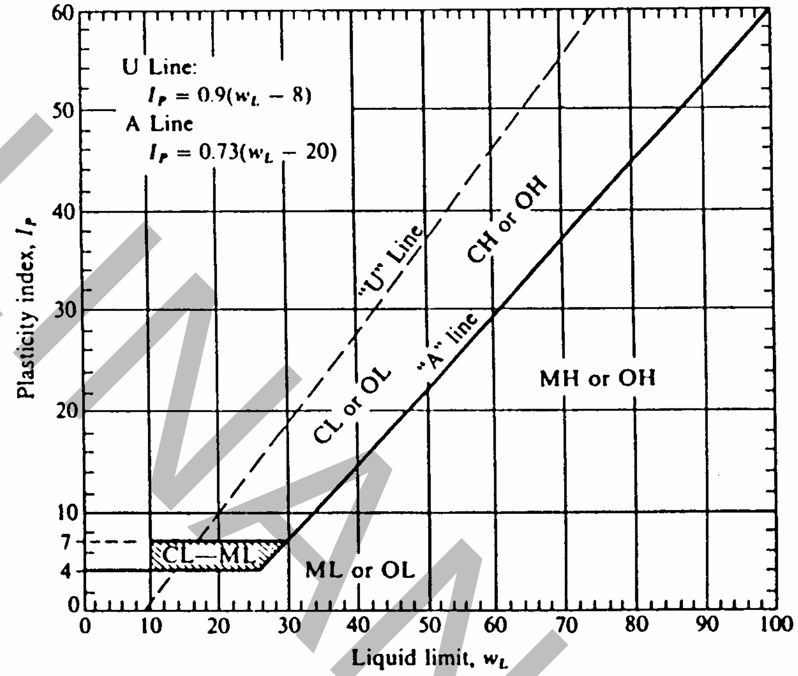
Table 11.1 - Unified Soil Classification System

Major divisions			Group symbols	Typical names	Laboratory classification criteria	
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP; More than 12 percent: GM, GC, SM, SC; 5 to 12 percent: *Borderline* cases requiring dual symbols‡	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting C_u or C_c requirements for GW
		Gravels with fines (Appreciable amount of fines)	GM† d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below A line or I_p less than 4 — Limits plotting in hatched zone with I_p between 4 and 7 are *borderline* cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above A line with I_p greater than 7
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting C_u or C_c requirements for SW
		Sands with fines (Appreciable amount of fines)	SM† d / u	Silty sands, sand-silt mixtures		Atterberg limits below A line or I_p less than 4 — Limits plotting in hatched zone with I_p between 4 and 7 are *borderline* cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above A line with I_p greater than 7

Table 11.2 - Unified Soil Classification System *(continued)*

Major divisions		Group symbols	Typical names
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	Silts and clays (Liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
	Highly organic soils	Pt	Peat and other highly organic soils

For all soils plotting nearly on A line use dual symbols, i.e., $I_p = 29.5$, $w_L = 60$ gives CH-OH or CH-MH. When w_L is near 50 use CL/CH, ML/MH. Take near as ±2 percent.



† Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when w_L is 28 or less and the I_p is 6 or less; suffix u used when w_L is greater than 28.

‡ Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

11.9 REFERENCES

English Language References

Reference **Publication**

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11.2 RICHARDS L.R., MILLAR P.J. and MARTIN G.R., *Rock Slope Stability*, Seminar on Bridge Design and Research, N.R.B. RRU, TC4, Wellington, Oct 1974.

11.3 TAYLOR P.W., *The Properties of Soils under Dynamic Stress Conditions with Application to the Design of Foundations in Seismic Areas*, Ph.D. thesis, University of Auckland, 1971.

11.4 UNITED STATES, DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, *Earth Manual*, 1965.

Additional References

Refer to Section 9 of this manual for additional *Indonesian Language* and *English Language* references.

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DIRECTORATE GENERAL OF HIGHWAYS
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

BRIDGE INVESTIGATION MANUAL

SECTION 12

DERIVATION OF DESIGN PARAMETERS

FEBRUARY 1993

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12. DERIVATION OF DESIGN PARAMETERS

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12. DERIVATION OF DESIGN PARAMETERS

12.1 INTRODUCTION

This section of the manual explains the mechanism for and derivation of the following design parameters :

- earthquake design parameters
- liquefaction potential
- slumping potential

12.2 EARTHQUAKE DESIGN PARAMETERS

The parameters required in the design of earthquake resistant bridge structures and in the assessment of foundation behaviour under earthquake conditions is detailed in the *Bridge Design Code (designated as the* ***Code*** *in subsequent text), Section 2.4.7, Earthquake Effects*.

12.2.1 Recommended Ground Acceleration

The *Recommended Ground Acceleration*, called the *Base Shear Coefficient C* in the *Code*, for each *Seismic Zone* show in *Code Figure 2.15* can be determined from *Code Figure 2.14* for a bridge *Period of Vibration T* equal to zero and appropriate soil type.

12.2.2 Recommended Design Ground Acceleration

The *Recommended Design Ground Acceleration* is obtained by multiplying the *Ground Acceleration* by the *Importance Factor I*, obtained from *Code Table 2.13*. That is,

$$\textit{Recommended Design Ground Acceleration} = C\ I \tag{12.1}$$

where ***C*** = *Base Shear Coefficient (Code Figure 2.14)* for the appropriate zone, period and site conditions (in this case use period T=0)

I = *Importance Factor (Code Table 2.13)*

12.3 LIQUEFACTION POTENTIAL

12.3.1 Introduction

For a bridge on a saturated sandy site the effects of liquefaction and the ensuing displacements can be so damaging that it may be totally destroyed.

The likelihood of liquefaction can be assessed by a number of methods. This section discusses the problem of liquefaction with reference to recent research into methods of determining liquefaction potential. A simplified method in common use is also given.

12.3.2 Liquefaction Mechanism

Granular materials generally tend to compact or densify under vibration. Hence when earthquake vibrations travel through a saturated granular deposit, the soil particles tend to compact. The contact forces between the soil grains are carried by a pressure increase in the more incompressible pore water. As a result intergranular contact is lost and the saturated granular deposit acts as a liquid mass with little or no shear strength.

Any bridge foundation supported by soil (or either footings or piles and especially abutments founded on approach embankments) would suffer large displacements. Liquefaction, however, rarely causes vibration damage to the structure itself because a weak liquefaction soil cannot transmit the earthquake forces to the structure. Instead damage to the structure will result from the total loss of support. Examples of this are :

- loss of bearing support resulting in large direct settlement or tilting
- loss of lateral support resulting in translation of footings or piles.

Reference is made to Figure 12.1 which shows in diagrammatic form the relationships between unidirectional stress and change in volume for loose, medium and dense sand. As shown, loose sand (with a void ratio greater than the critical void ratio) will continue to compact under a unidirectional load at increasing rates of strain (that is, with increasing load). Under saturated conditions as such sand would liquefy.

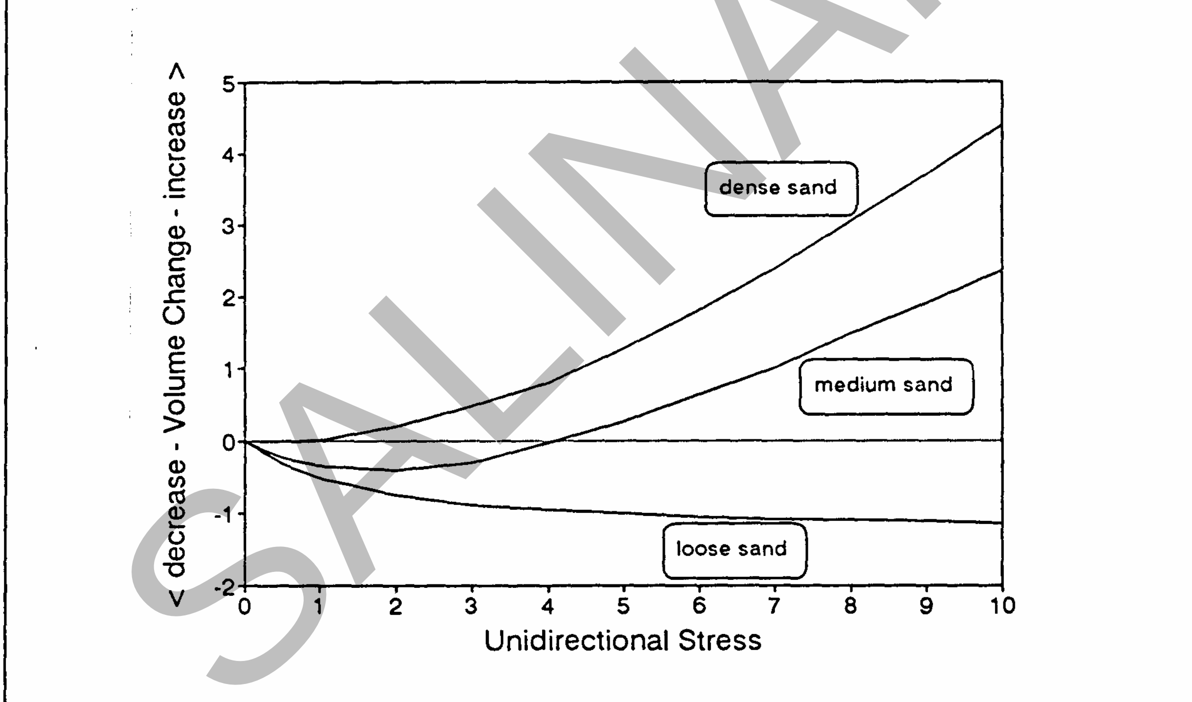

Figure 12.1 - Volume Change of Sand under Unidirectional Loading

A medium sand compacts at small strains and dilates at large strains under unidirectional load. Dense sand tends to dilate under unidirectional loading, regardless or the magnitude of the strain. Hence the risk of medium sands liquefying under unidirectional load is much less than for loose sands and dense sands cannot liquefy under unidirectional load.

However, under cyclic loadings (noting here that earthquake loading has a random cyclic nature) progressive compaction will occur, even for very dense sands. Hence given a sufficient number of cycles of loading any saturated sand could liquefy, but the number of cycles required for very dense sands is likely to be well in excess of that experienced during a major earthquake.

Field and laboratory studies on liquefaction of sand (Reference 12.1) have established the following factors to influence that liquefaction potential of a soil deposit.

- **Soil Particle Size**

  Cohesionless soils whose particle size distribution falls within the zones described in Figure 12.2 are the most likely soil type in which liquefaction may occur.

- **Relative Density**

  Looser sands tends to liquefy more easily than dense sands. The relative density of a sand can be directly measured or more often related to the ***N*** value (in blows/300 mm) as measured by a standard penetration test.

- **Confining Pressure**

  The higher the confining pressure the higher the required cyclic stress to induce liquefaction.

- **Intensity of Shaking**

  For a given soil under a certain confining pressure, its vulnerability to liquefaction depends on the magnitude of stresses and strains induced by the earthquake, these are related to the intensity of the ground shaking.

- **Duration of Shaking**

  The likelihood of the onset of liquefaction in a certain soil subjected to a certain intensity of shaking depends on the duration of that shaking.

12.3.3 Selection of Horizontal Ground Acceleration Value

The intensity of shaking depends on the magnitude of the earthquake and the distance of the site from the epicentre. Acceleration levels at the epicentre of the greatest earthquake recorded are estimated to be in excess of gravity (***1.0 g***) and under such an acceleration it is likely that even a dense sand could liquefy. However, this intensity drops rapidly away from the epicentre. The probability of a site being at the epicentre of an earthquake is much smaller than of the site being at a sufficient distance away from the epicentre for acceleration values to be very much lower. Following consideration of the recorded seismicity of the area and the above principles, the recommended horizontal ground acceleration values for determination of liquefaction potential are the same as those which are determined as outlined in Section 12.2.2.

The acceleration values shown on Figure 12.3 are maximum ground surface acceleration values. These values, together with the standard penetration resistance value, enable an assessment to be made as to whether the ground is or is not likely to liquefy under any conditions.

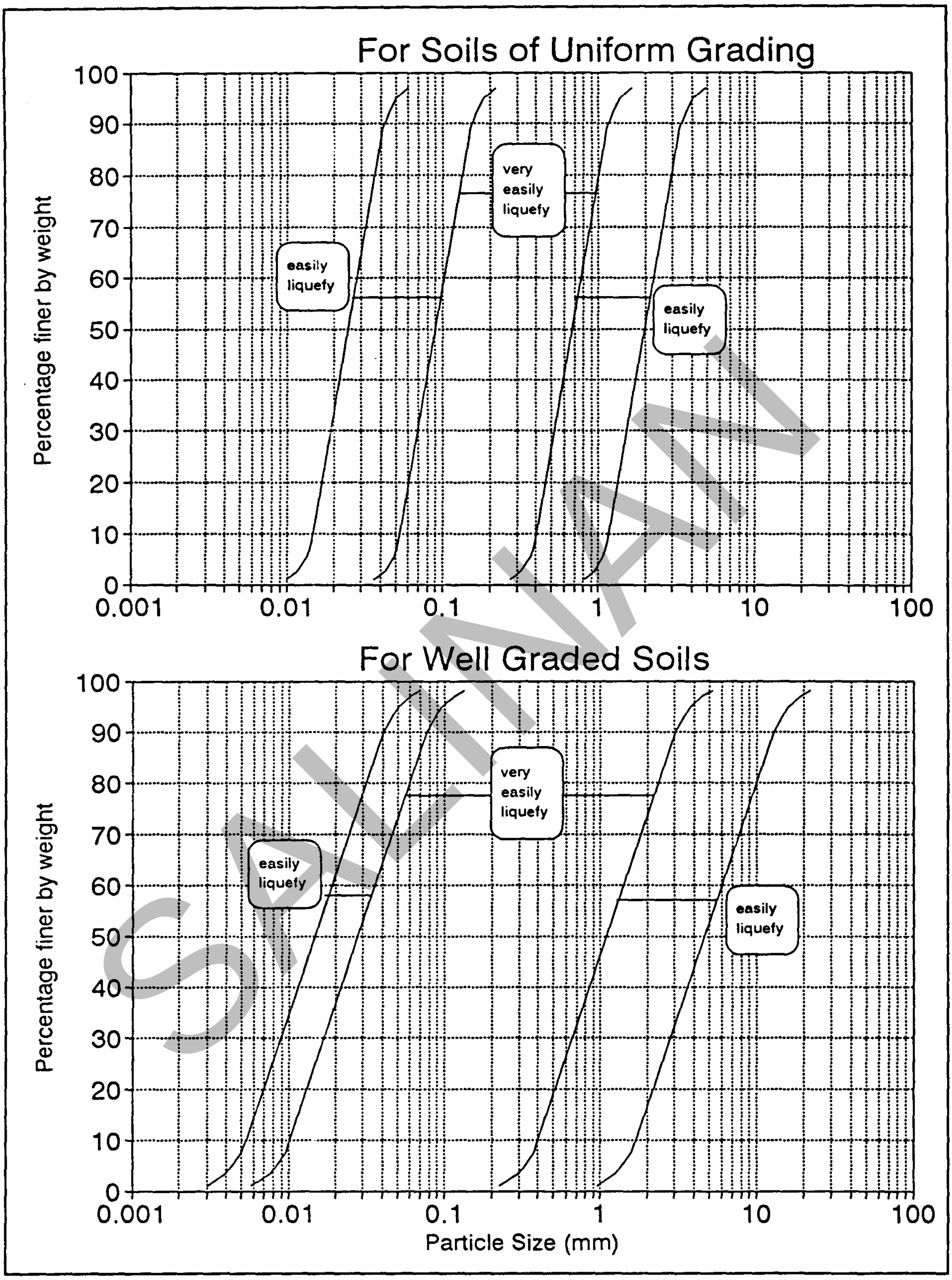

Figure 12.2 - Liquefaction Potential of Sands

It must be understood that if a site is hit by a large earthquake whose epicentre is immediately beneath the site, the maximum horizontal ground acceleration value is likely to be much higher than the values given in the *Code, Section 2.4.7*. Hence the risk of structural damage, slope stability failure and (if the soils present such a risk) liquefaction, is very real indeed. Therefore a careful analysis of the probability of such an occurrence should be made for major structures in order to assess the level of risk which can be accepted depending on the importance of the bridge and the possible consequences if it became unserviceable as a result of such an earthquake.

12.3.4 Liquefaction Potential Assessment

a. Simplified Method Based on Chinese Building Code Data

Seed (Reference 12.2) reported the use of a correlation between standard penetration resistance N_c separating liquefiable from non-liquefiable conditions to a depth of about 15 m based on the following equation :

$$N_c = \bar{N} [1 + 0.125 (d_s - 3) - 0.05 (d_w - 2)] \qquad (12.2)$$

where d_s = depth of sand layer (m)

d_w = depth of water table (m)

$\bar{N}$ = function of shaking intensity - see Table 12.1 for values

Table 12.1 - Values of $\bar{N}$ vs Modified Mercali Intensity

Modified Mercali Intensity Scale	Blows/300mm $\bar{N}$	Modified Mercali Felt Intensity
VII	6	strong shaking, difficult to stand up, damage to poor buildings, serious cracking (ground acceleration = approx. 0.1g)
VIII	10	severe shaking, damage to masonry building, chimneys come down, destructive (ground acceleration = approx. 0.2g)
IX	16	violent shaking, general panic, serious damage, ground crack up, devastating (ground acceleration = approx. 0.4g)

Table 12.1 shows that N_c is based on *felt* intensities of shaking.

Based on the above correlation between felt intensities (that is, Modified Mercalli Scale, MM) and ground acceleration Figure 12.3 has been compiled for ease of use.

The soil parameters required for this particular analysis are listed in Table 9.3 together with the investigation methods available to obtain these parameters.

The liquefaction potential of a site may be assessed following Steps 1 to 5 in Table 12.2.

Table 12.2 - Assessment of Site Liquefaction Potential

Step	Procedure
Step 1	Determine a suitable level of maximum ground acceleration. The *Recommended Design Ground Acceleration* determined as outlined in Section 12.2.2 should be used.
Step 2	Plot the grain size distribution (as percentage finer than a certain sieve size) on Figure 12.2 to assess the possibility of liquefaction. If the material grading shows a large proportion of silt/clay or gravel, the possibility of liquefaction would be slight.
Step 3	The ground water table level is noted. The higher the water table the higher the risk. A low water table does not exclude the risk of liquefaction which may occur at depth causing movements at the ground surface. Generally, however, this risk would be small unless the excess pore pressure developed in the liquefied layer dissipates upwards causing liquefaction of overlying layers or boils at the ground surface.
Step 4	Results of standard penetrations tests are standardised on two counts. Firstly the SPT blow counts (***N***) obtained in the field should be normalised to an effective overburden pressure of 1 ton/sq.ft (or about 100 kPa) by using the equation : $N_1 = C_N \times N$ where C_N is shown on Figure 12.4. Secondly it should be noted that all data referred to by Seed et al in their publication (Reference 12.3) are based on SPT tests using the rope and drum procedures with two turns wrapped around the rotating drum. The energy delivered by a hammer controlled by rope and drum is only about 60% of that delivered by a free falling weight for 2 turns (or 40% for 3 turns). Therefore if the SPT's were carried out using a trip hammer which allows free fall the results should be increased by a factor or 1.6 before carrying out the overburden correction (C_N) discussed above.
Step 5	The corrected N_1 values can be plotted on Figure 12.3 to assess liquefaction potential. For the appropriate depth and standard penetration test result Figure 12.3 indicates the ground acceleration threshold beyond which liquefaction must be considered likely.

If liquefaction is found to be likely to occur then either :

- Design measures should be implemented to minimise damage in the event of liquefaction, *or*
- Soil densification measures should be implemented to reduce the risk of liquefaction.

It should be noted that on Figure 12.3 and Figure 12.4 the standard penetration test results have been corrected for variation in overburden pressure using the relationships derived from the work of Gibbs and Holtz.

b. Method Based on Cyclic Stress Ratio Analysis

The *Cyclic Shear Stress Ratio* is defined (Reference 12.3) as :

$$\left(\frac{\tau}{\sigma_0'}\right)_{ave} = 0.65\,\frac{a_{max}}{g} \times \frac{\sigma_0}{\sigma_0'} \times r_d \qquad (12.3)$$

where a_{max} = maximum ground acceleration at site

σ_0 = total overburden pressure on sand layer in question

σ_0' = effective overburden pressure on sand layer

r_d = stress reduction factor, being 1 at ground surface and 0.9 at about 10 m depth

g = acceleration due to gravity

The calculation steps are similar to those presented in Section 12.3.4.a above for the normalisation of SPT N values to N_1, that is, follow Steps 1-5(b) in Table 12.3.

Table 12.3 - Cyclic Stress Ratio Method

Step	Procedure
Steps 1 to 4	Follow Steps 1 to 4 in Table 12.2.
Step 5(b)	Calculate the cyclic stress ratio as indicated above and enter Figure 12.5 to assess liquefaction potention noting that a *design* earthquake magnitude must be selected (a reasonable assessment would be; Zone 1, M = 8.0; Zones 2 and 3, M = 7.5; Zone 4, M = 7.0)

Note that for *silty* sites the resistance to liquefaction is greater.

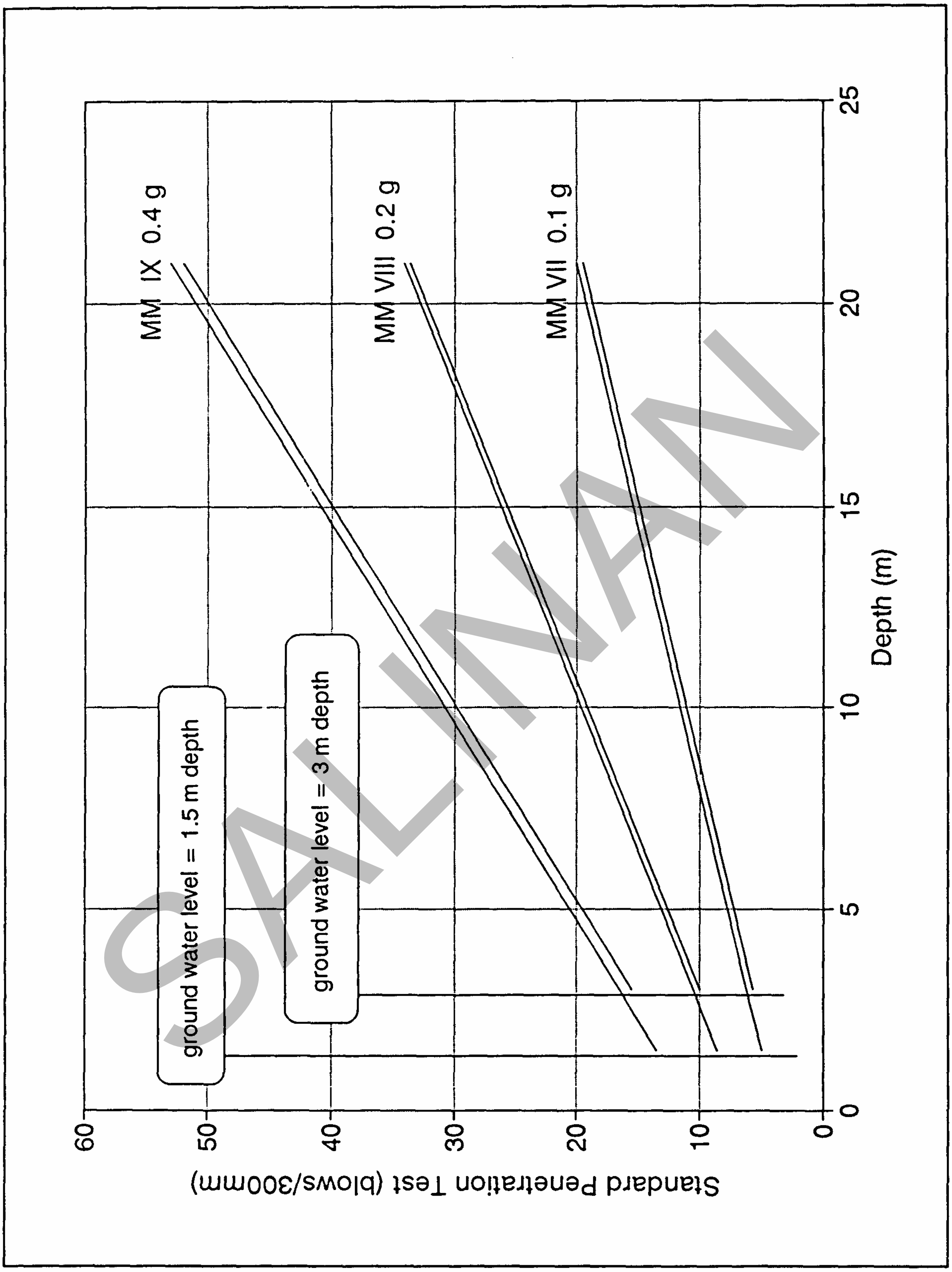

Figure 12.3 - Penetration Resistance Limits for Determination of Liquefaction Potential

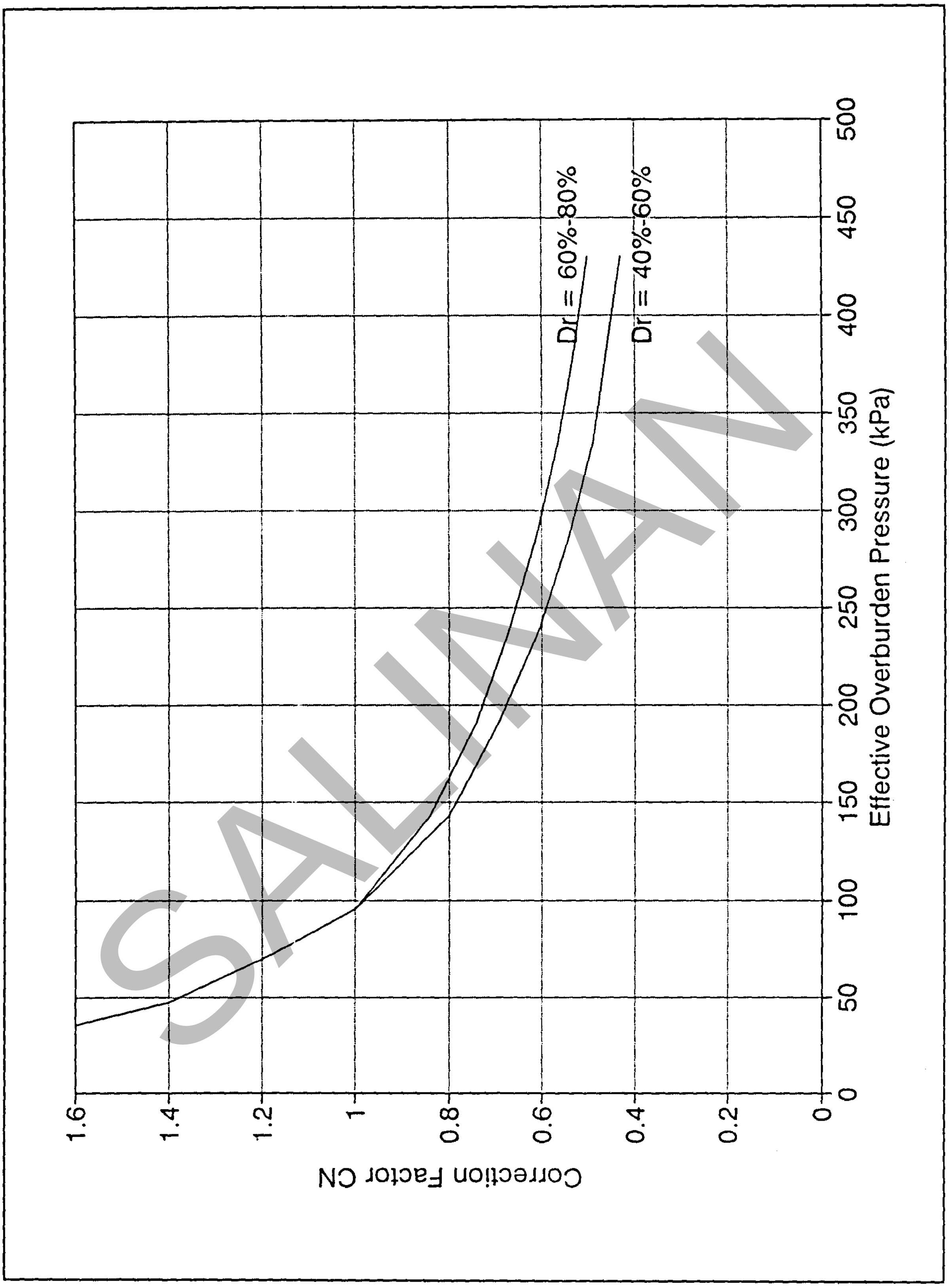

Figure 12.4 - Recommended Curves for Determination of C_N

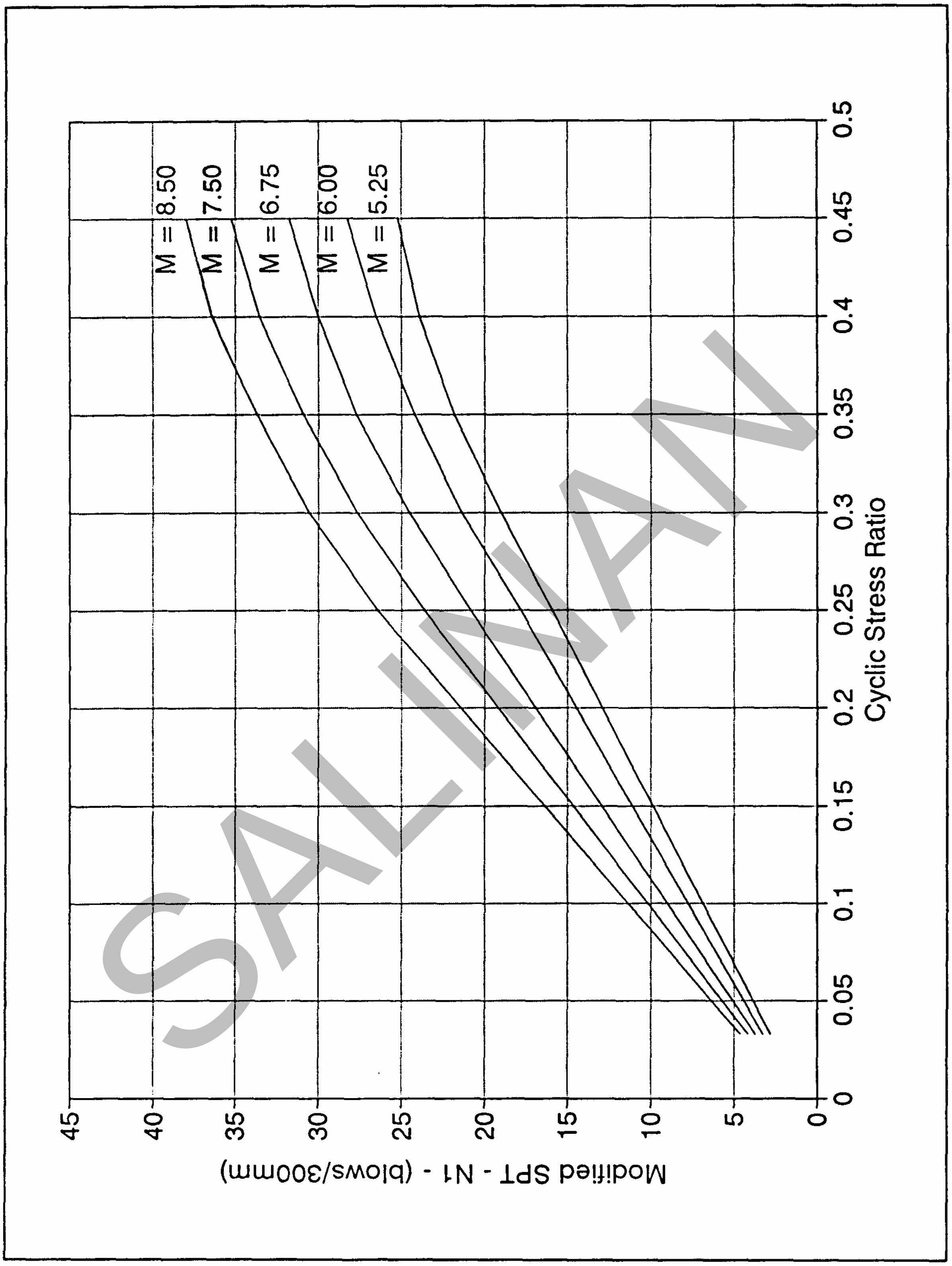

Figure 12.5 - Chart for Evaluation of Liquefaction Potential for Different Magnitude Earthquakes

12.4 SLUMPING POTENTIAL

12.4.1 Mechanisms of Slumping

Slumping is a rapid failure of a man made embankment or natural slope resulting in large settlements which is often accompanied by a flow resulting in much flatter batter slopes. Observations of slumping by earthquakes have shown vertical displacements in the order of ½ to 2 metres.

Slumping of embankments caused by earthquake vibrations may be attributed to mechanisms :

- **from within the embankment**
  - for a non-cohesive embankment the soil mass may become mobile (that is, lose shear strength) under the earthquake shaking and slump as a consequence. Slumps can also be caused by slips parallel to the surface of natural slopes, sliding on firmer material beneath. Localised liquefaction may cause slumping in an embankment or slope where the ground water table is reasonably high.
  - for a cohesive embankment the shearing stresses generated by the shaking may momentarily cause the soil to yield, leading to displacement of the soil mass. These displacements often appear as multiple slips occurring both in the embankment soil and the soil below.
- **from beneath the embankment**
  - if the embankment is placed on top of a sensitive cohesive material which would lose strength under large shearing stresses induced by the earthquake, the material may weaken and plastic flow of the soil beneath the embankment could result. This may be either a lateral flow (leading to longitudinal cracks forming in the embankment) or a longitudinal flow leading to slumping of the slope batter of the approach embankment adjacent to a bridge.
  - an embankment placed on top of a saturated sand may suffer similar damage if the sand beneath the embankment liquefies during an earthquake.

12.4.2 Slumping in Non-Cohesive Soils

The extent of damage due to slumping failure may vary widely depending on the mechanisms of failure.

a. Where Liquefaction is Likely

The worst type of failure in a non-cohesive soil is a slumping failure caused by liquefaction. Where liquefaction is likely to occur the foundation soils must be strengthened (densified) if the risk of such failure is to be minimised.

b. Where Liquefaction is Unlikely

Where liquefaction is unlikely, slumping may be caused by a series of instantaneous slip failures under transient earthquake accelerations which would result in smaller displacements.

- **Permanent Bridges**

  In the case of *permanent bridges*, it will usually be sufficient to design the batter slope of the embankment or natural ground for a factor of safety of 1.5 under static loadings. In the event of an earthquake the factor of safety of the slope may momentarily drop below 1.0. However, the permanent displacements induced do not constitute a failure and repair work to build up the embankment would be simple and inexpensive.

- **Important Bridges**

  In the case of an *important bridge* or a vital *permanent bridge*, it may be imperative to minimise slump displacements. This can be done by increasing the factor of safety in the slope design to 3.0, or by analysing the slope stability for an added horizontal acceleration to twice the earthquake ground design accelerations.

c. Bridge/Embankment Arrangement

A bridge end span should not be supported by an embankment alone if the embankment is constructed over non-cohesive soils unless the displacements expected can be tolerated by the bridge superstructure. Even if liquefaction does not occur during an earthquake, settlements of the embankment due to densification in the sand beneath could be sufficient to cause severe damage to the bridge structure unless precautions are taken. Repairs would consist of jacking the embankment end back to level and reconstructing the embankment below. Continuous bridge superstructures should not be used in such situations.

12.4.3 Slumping in Cohesive Soils

For a cohesive soil the most damaging slumping failure is acused by the decrease in strength of a sensitive clay layer within the embankment foundation or beneath the slope in natural ground.

a. Soil Sensistivity

To assess the sensitivity of the soil, shear vane tests could be carried out on the remoulded material. The sensitivity is defined as :

$$S_t = \frac{\textit{undisturbed strength}}{\textit{remoulded strength}} \qquad \textbf{(12.4)}$$

b. *Quick Clay*

For values of S_t greater than **8**, the clay is termed a *quick clay*, and excessive displacements can be expected to occur during shaking and extensive failure of the embankment slopes (or slopes in natural ground) may result.

c. Determination of Safe Batter Slopes

Where such failures are unlikely, slumps of smaller magnitude can be caused by localised momentary yielding and the factors of safety given in Section 12.4.2.b can be applied in determining safe batter angles for the slope.

d. Bridge/Embankment Arrangement

The notes on bridge/embankment arrangement in Section 12.4.2.c also apply to cohesive soils.

12.5 REFERENCES

English Language References

Reference **Publication**

12.1 SEED H.B. & IDRISS I.M., *Simplified Procedure for Evaluating Soil Liquefaction Potential*, A.S.C.E., Journal of the Soil Mechanics and Foundation Division, Vol. 97, No. SM9, September 1971, pp 1249-1273.

12.2 SEED H.B., *Soil Liquefaction and Cyclic Cobility Evaluation for Level Ground during Earthquakes*, A.S.C.E., Journal of the Geotechnical Division, Vol. 105, No. GT2, February 1979, pp 201-266.

12.3 SEED H.B., IDRISS I.M., & ARANGO I., *Evaluation of Liquefaction Potential using Field Performance Data*, A.S.C.E., Journal of the Geotechnical Division, Vol. 109, No.3, March 1983, pp 458-482.

12.4 MAKDISI F.I. & SEED H.B., *Simplified Procedure for Evaluating Embankment Response,* A.S.C.E., Journal of the Geotechnical Division, Vol. 105, No. GT12, December 1979, pp 1427-1434.

Additional References

Refer to Section 9 of this manual for additional *Indonesian Language* and *English Language* references.

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